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Urban Storm Runoff and Combined Sewer Overflow Pollution

Sacramento, California

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~~Feuerstein, D. L.~~ King, R. W. ~~Grimm, A.~~

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Necessary data are not available in most areas to determine or predict distributions of storm water runoff and combined sewage flows and pollutant concentrations. To alleviate this problem, a procedure was developed to permit rapid, economical, and accurate assessments of system performance based on 3 different and important water quality criteria. The first criterion establishes an absolute maximum pollutant concentration which cannot be exceeded. The second criterion establishes an acceptable distribution of pollutant concentration by specifying the greatest frequency of occurrence for a particular concentration value. The third criterion establishes the maximum acceptable excession frequency for a particular pollutant frequency. A computer model was then used with a modified rational runoff method, to perform preliminary design tasks and weigh alternatives to the designs. The least costly system developed for the Sacramento area incorporated existing and future combined sewers for conveyance with treatment for the combined wastewater, since storm runoff has a chemical composition similar to that of raw sewage. (Lowry-Texas)

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Mark V. Lowry

Texas Univ, Austin

URBAN STORM RUNOFF
AND
COMBINED SEWER OVERFLOW POLLUTION

Sacramento, California

by

Envirogenics Company
A Division of
Aerojet-General Corporation
El Monte, California

for the

ENVIRONMENTAL PROTECTION AGENCY

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ABSTRACT

A general method was developed to assess, primarily from readily available precipitation and wastewater quality data, the extent of water pollution occurring from storm water runoff and combined sewer overflows in an urban area, and is applied to Sacramento, California. Systems for the control and treatment of these wastewaters are developed and evaluated.

The least costly system to adequately protect the receiving waters from storm water runoff and combined sewer overflows would retain the combined sewers for the conveyance of combined sewage during wet-weather flow conditions. Facilities would also be required for the treatment of existing separated storm water flows. Total annual cost for this system was estimated to be $6.99 million. A slightly more costly system ($7.09 million) incorporating complete sewer separation of sanitary sewage and storm water runoff is recommended to the City of Sacramento. The similarity in annual costs for the separated sewer and the combined sewer systems results from the requirement for major enlargement of the existing combined sewer system to adequately convey anticipated combined sewage flows. In areas where existing combined sewer capacities would not be grossly inadequate, the separation of combined storm water runoff and sanitary sewage flows to achieve receiving water quality objectives would appear unwarranted, due to the high cost of constructing new conveyance facilities and the probable requirement to treat separated storm water runoff, since its quality is not substantially different from that of sanitary sewage.

This report was submitted in fulfillment of Project Number 11024 FKM, Contract Number 14-12-197, under the sponsorship of Water Quality Research, Environmental Protection Agency.

Pursuant to Executive Reorganization Plan No. 3 of 1970, effective December 2, 1970, and Environmental Protection Agency Orders Nos. 1110.1 and 1110.2, all references to the Federal Water Quality Administration herein shall be to the Environmental Protection Agency, Water Quality Research.

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Section I

CONCLUSIONS AND RECOMMENDATIONS

This program was directed to the development of a general method for determining the extent of pollution resulting from storm water runoff and combined sewer overflows occurring in an urban area, and the application of this method to the City of Sacramento, California. Whereas the conclusions and recommendations, which are primarily based on conditions and factors pertaining to Sacramento, may not be general in their application to other areas, the methods developed in this program to assess the extent of pollution from storm water runoff and combined sewer overflows are believed general in nature.

CONCLUSIONS

The necessary data are not available in the Study Area or most other areas to determine or predict distributions of storm water runoff and combined sewage flows and pollutant contents, and distributions of the corresponding receiving water characteristics. Most, if not all, of the characterization of storm water runoff and combined sewage that has been performed and reported is totally unsuitable for predicting in a quantitative manner the temporal distributions of flows and compositions of storm water runoff. The best that these data can provide are qualitative indications of the magnitudes and range of expected variations in the characteristics.

Because the development of specialized data acquisition and measurement programs are both time consuming and costly, a method of assessment is needed that can rely upon readily available, ubiquitous historical data. Long-term records, in contrast to data collected during short-term programs, provide a greater assurance that extreme values or occurrences, for which storm water runoff and combined sewage control systems must be designed, will be noted and accommodated. *A method was developed in this program that provides a rapid, economical, and accurate assessment of the extent of pollution resulting from storm water runoff and combined sewage overflows and the effectiveness of systems in controlling the pollution.*

Evaluation of system performance should be based, not on single values or sets of water quality criteria that are applicable for all occasions at all times, but on water quality criteria that recognize and reflect naturally occurring and tolerable distributions of water quality and their expectancies. *The procedure developed in this program permits performance assessment on the basis of three different and important water quality criteria* -- maximum value, cumulative distribution, and excession frequency. The first criterion establishes an absolute maximum pollutant concentration that cannot be exceeded at any time. The second criterion establishes an acceptable distribution of pollutant concentration by specifying the greatest frequency of occurrence for a particular concentration value. The third criterion establishes the maximum acceptable excession frequency for the occurrence of a particular pollutant concentration.

The computer program developed in this program for performing the preliminary design of gravity and force main sewer networks, including separate sanitary, separate storm water runoff, and combined sewage collection and conveyance facilities, provided rapidly and economically all of the design details necessary for the accurate preparation of cost estimates. Data input to the program consists simply of the flow distribution networks and the assignment of the following appropriate values to each junction in the network: resident and transient populations, storm runoff coefficient, individual drainage area, ground surface elevation, and distance to next downstream junction. Output from the program provides the peak flow, diameter, slope, invert elevation, and depth below grade for every pipe in the network. With the aid of this computer program, a number of conveyance alternatives for several different storm recurrence intervals can be designed and their costs estimated to permit a comprehensive evaluation based on many alternatives.

The modified rational runoff method developed in this program provided realistic estimates of peak storm water runoff flow rates even though the total drainage area to which it was applied exceeds ten square miles. The method aggregates the individual hydrographs arriving from upstream contributing sewers to establish the peak flow and the corresponding time of concentration to be used for the design of the immediately downstream sewer.

Due to the high cost of constructing new collection and conveyance systems and the poor quality of storm water runoff, the separation of combined storm water runoff and sanitary sewage flows to achieve receiving water quality objectives appears unwarranted, except perhaps in those areas where existing combined sewer capacities are grossly inadequate. The concentrations of pollutants in separated storm water runoff are sufficiently large, and not substantially different from those in sanitary sewage, to require some degree of treatment prior to their discharge to receiving waters under most conditions.

Separation of sanitary sewage and storm water runoff in combined sewer systems by using a force main for the transport of the sanitary wastes is substantially more expensive than by employing an additional gravity-flow system. In the portion of the Study Area underlain with combined sewers, the estimated annual construction, operation, and maintenance cost of a system embodying a completely pressurized network extending from each individual service connection to the terminus outfall is in excess of $15 million. If the small diameter gravity laterals, which contain most of the service connections and which would be abandoned and unused under the aforementioned force main system since they are either inadequate or unnecessary for transporting storm water runoff, were utilized to convey sanitary wastewaters to common grinder-pump stations for injection into the pressurized system, the estimated total annual cost of this force main system is about $11 million. A further 20-percent reduction in system costs would result if the dual-pipe system used in this program were replaced by a single-pipe network. An equivalent gravity system, however, costs less than $2 million.

For those alternative systems containing high-rate treatment processes, such as mechanical screening and dissolved air flotation, the least costly facility always resulted when the treatment process was combined with a holding pond whose capacity was matched with that of the process to provide an optimally sized facility. The holding pond provides mainly peak flow attenuation, but also some pollutant removal capability as well. Reduction of instantaneous peak flows results in both smaller downstream treatment facilities with improved utilization factors and lesser instanteneous pollutant loadings and effects on the receiving waters. Determination of the required capacities of ponds to contain storm water runoff and combined sewer overflows associated with specific storm recurrence intervals must be based on temporal distributions of both inflow and outflow.

The least costly system for the Study Area that will adequately provide for the protection of the receiving waters from the discharge of storm water runoff and combined sewer overflows retains the combined sewers for the conveyance and provides for the treatment of the combined wastewater. The system would provide for two separate conveyance and treatment systems in the Study Area--one in the existing combined sewer area and the other in the existing separated sewer area. The Combined Area system is comprised of an augmented existing combined sewer collection system for the conveyance of sanitary sewage from the Study Area and storm water runoff from the Combined Area to a treatment facility with a surface reservoir, a dissolved air flotation unit, and a chlorination station. In the Separated Area, the system consists of an augmented existing storm water runoff collection network and a new storm water interceptor sewer for conveyance of the storm water runoff to a common treatment facility comprised of a holding pond, a dissolved air flotation unit, and a chlorination station.

RECOMMENDATIONS

To protect the Sacramento River from the discharge of storm water runoff and combined sewer overflows from the total combined sewer system area until the year 1992, *it is recommended that the City of Sacramento select a system which provides for the complete separation of sanitary sewage and storm flows and provides for two separate storm water conveyance and treatment systems.* Storm water runoff from the existing combined sewer service area would be conveyed in a new gravity sewer system to a treatment facility containing a holding pond, a dissolved air flotation (or mechanical screening) unit, and a chlorination station. A similar facility would treat storm runoff conveyed from the existing separated sewer areas in augmented storm sewers. Sanitary sewage from the total area would be conveyed to the present sewage treatment plant in the existing combined sewer system. Although this system has a greater total cost than one retaining combined sewers for combined sewage, it provides a much greater opportunity for extramural financial support and would cost the local participants annually about 100 times less if total maximum funding eligibility under existing laws could be realized.

In an attempt to reduce further the total cost of acceptable water pollution abatement systems, *it is recommended that an investigation be conducted to determine the merits of providing different retention and treatment subsystems for different portions of the storm water runoff and combined sewer overflows.* Because usually both the flow rates and pollutant concentrations of storm water runoff peak during the initial periods of rainfall events and decline rather rapidly thereafter, the separation and treatment of a portion of the flow for a fraction of the runoff time could produce substantial reductions in pollutant loading to receiving waters at relatively low costs. The performance of the recommended investigation would require a detailed quantitative analysis of temporal wastewater flow and quality distributions and corresponding receiving water distributions, and could be accomplished by modifying the procedures developed and reported herein.

During the conduct of this program to develop a method for evaluating systems for the control of pollution from storm water runoff and combined sewage overflows, it was necessary to determine the short-term or hourly variations in quality of the storm water runoff that would be expected to occur in the Study Area. In the absence of a known method, these storm water compositions were estimated by means of a model developed specifically in this program which relates pollutant concentration to the time and rate of pollutant accumulation on the ground surface and the rate of storm water runoff. The model is simple and rationally derived, but only extremely limited data were available to establish its exact form and values of coefficients. Because the accurate prediction of instanteneous, or hourly, storm water runoff pollutant concentrations is required in the quantitative evaluation of storm water runoff and combined sewer overflow treatment systems, *it is recommended that a study be undertaken to provide the necessary information for the verification or modification of the storm water quality model format and for the determination of the coefficient values.*

The water quality requirements criteria for storm water runoff and combined sewage treatment systems almost always depend upon the quality and flow characteristics of the receiving waters, which in turn are totally influenced by upstream conditions and facilities. Whereas analysis and evaluation of systems within a particular area can be performed and the best system established by the methods presented herein, the quantitative effect and inter-relations of conditions outside the area that may markedly affect the analysis are not evaluated. *It is recommended, therefore, that the procedures developed herein be expanded in application to include consideration of all wastewater discharges, including storm water runoff from urban and rural areas, combined sewer overflows, municipal sewages, industrial wastewaters, and irrigation return waters, to the receiving waters of the drainage basin.* The analysis of an entire drainage basin system would delineate the extent to which each discharge contributes to the pollution burden of the receiving waters under various wastewater management systems. This approach would assure the best allocation of resources for the protection of the waters in the drainage basin.

The performance of the high-rate treatment processes--dissolved air flotation and mechanical screening--which provided for the conditions of this study acceptable systems at least cost was based on limited laboratory-scale and pilot-plant studies. On the basis of latest available information which indicates that mechanical screening should be less costly than and as effective as dissolved air flotation in removing wastewater pollutants, *it is recommended that a program be undertaken to demonstrate the efficacy of the mechanical screening process described herein to adequately treat separated storm water runoff and to establish operating parameters and realistic capital, operating, and maintenance costs.*

Section II

INTRODUCTION

Man's awareness of his environment is growing pronounced and his concern intense as he becomes more populated, more urbanized, and more affluent. The need for a safe and potable water supply and the resulting degradation of that water by its use was most likely recognized far sooner than similar considerations of air and land resource management, simply because water pollution has manifested itself in the loss of human life, the loss of fisheries and wildlife resources, the loss of irrigable crops, the loss of usable water supplies, and the loss of aesthetic values.

Emphasis was placed initially on the control of sanitary sewage for the protection of public health and safety. As a result, sanitary wastes were collected in an underground sewer network, conveyed to some distant location, and disposed of untreated. Wherever the disposal occurred upstream of water supply intakes, natural diminution by dilution and degradation and subsequent water supply treatment usually combined to afford adequate protection. The runoff from rainfall was carried on the surface or collected and transported in separate storm sewers; or in many cases, the drainage was conveyed in sewers designed for the simultaneous combined transport of both storm water runoff and sanitary sewage.

As populations expanded and had greater occasion to come in contact with their surroundings, the presence of unsightly and malodorous sludge deposits resulting from the discharge of untreated sewage became unacceptable, and primary sewage treatment was provided. As the burdens on the assimulative capacity of receiving waters escalated due to the ever increasing amounts of degradable soluble materials, the oxygen resources of the receiving waters became overtaxed and fish and wildlife resources were affected and septic conditions obtained. This condition was alleviated by the provision of secondary sewage treatment, but as the demands on the available resources have grown more critical, emphasis has shifted toward advanced waste treatment processes for the removal of nutrients and biostimulants to retard eutrophication of receiving waters and for the removal of refractory organic and inorganic substances and toxicants for higher reuse of the renovated water.

With the accomplishment of more complete control of municipal and industrial wastewater discharges, other sources of water pollution have taken on greater significance in their contribution to the overall burden and are attracting attention and concern. Two such sources originating within urban areas are combined sewer overflows and separated storm water runoff.

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Whereas combined sewers may provide adequate capacity to carry peak storm flows, which are generally several orders of magnitude greater than the accompanying sanitary flows, sewage treatment facilities do not have the same hydraulic capacity. Therefore during periods of precipitation, the sewage treatment facility may be unable to cope with the combined flows so that the excess is diverted from the treatment plant and discharged directly to a receiving water or water course. These combined sewer overflows are of particular concern because they result in the bypass to the environment of untreated sewage and present a potential public health hazard.

Both the quantity and quality of separated storm water runoff from an urban area are highly variable and transient in nature, being dependent upon meteorological and climatological factors, hydraulic characteristics of the surface and subterranean conduits, and on the nature of the antecedent period. The pollutants contained in urban storm water runoff and drainage include street litter, spent foliage, dust and debris from airborne fallout, eroded soil, animal excreta, fertilizer, insecticides, ice-control chemicals, and many other chemicals and substances. Also, storm sewers in urban areas often receive cooling waters from refrigeration systems, wastewater from air pollution control operations, and sanitary and industrial wastewaters through illegal connections.

PROGRAM NEED

The need to investigate the storm water runoff and combined sewer pollution problem and to determine its extent and contribution to the total water pollution burden was amplified in 1965 by the U. S. Public Health Service in a report (Ref. 1) that pointed out the widespread use of combined sewer systems in the United States. A subsequent study (Ref. 2) conducted by the American Public Works Association for the Federal Water Pollution Control Administration in 1967 disclosed that there were 1,329 municipalities representing 54 million persons in the United States served totally or partially by combined sewers, and that about 70 million persons were served by separate sanitary sewers. The report further estimates that around 60 to 65 million persons reside in areas served by separate storm sewers, which were depicted as potential sources of water pollution. In 1968, another study (Ref. 3) was conducted by the American Public Works Association for the Federal Water Pollution Control Administration to determine the factors in the urban environment that contribute to the pollution of urban storm water runoff and to determine methods of limiting this source of pollution.

This study, presented herein, is directed at the next logical step; namely, the development of a method to assess the extent of the water pollution resulting from storm water runoff and combined sewer overflow and the description and cost of suitable facilities to satisfy specified receiving water quality criteria.

SCOPE AND OBJECTIVES

To provide adequate and economical solutions to storm water runoff and combined sewer overflow problems, careful analysis and evaluation of various alternative systems must be made relative to their total impact on the environment and the community. This program was aimed at the development of a general method for ascertaining the extent of pollution occurring as the result of storm water runoff and combined sewer overflows, and the synthesis, design, costing, and effectiveness of various alternative water pollution abatement systems. The method was directly applied to the City of Sacramento, California, which is characteristic of many older communities in the nation where the original sewer system, usually in the downtown or business district, was developed on combined sewers. Areas subsequently annexed to the City were provided with separated storm and sanitary sewers. The study area thus contains both combined and separated sewers, necessitating the integration of both systems in terms of their total effect upon the receiving waters in order to measure the overall effectiveness of any alternative system.

Seven different conceptual candidate systems were considered for the City of Sacramento that were believed to encompass, at least in principle, most feasible solutions applicable elsewhere in the nation. Two systems were based on the total separation of sanitary sewage from storm water flow--one by the construction of either a sanitary or a storm gravity sewer and the other by the collection and transport of sanitary wastewater in small diameter force mains. Storage was utilized for two systems, which in combination with other system components, provides either surface ponding or near-surface underground facilities. Two systems employed high-rate treatment processes, namely mechanical screening and dissolved air flotation. The last system was based on the use of large stabilization ponds.

A complete alternative system capable of handling all of the wastewaters--domestic, commercial, and industrial sewages and storm water drainage--generated within the study area, in addition to those transported through the study area, consists of adequate collection and conveyance, storage (or no storage), treatment (or no treatment), and disposal components. The various combinations of these components, when coupled with different storm recurrence interval design criteria and the realities of available storage pond and treatment facility locations, present an extremely large number of alternative water pollution control systems for analysis and evaluation.

Although the City of Sacramento was selected in this study, the program was designed and conducted to provide applicability to sewer systems throughout the nation with similar characteristics. Specific objectives of the program were:

> To provide sufficient hydrologic, meteorologic, demographic, geographic, and land-use bases for design of

alternative systems. Wastewater quantities and qualities were established by field sampling and analysis.

To develop preliminary designs for each of the alternative systems in sufficient detail to assess their ultimate cost.

To estimate costs for the construction, operation, and maintenance of the alternative systems.

To develop a method for assessing the performance and effectiveness of candidate water pollution control systems in meeting specified receiving water quality objectives.

To develop the cost-effectiveness of alternative systems, at least as pertains to the City of Sacramento.

Section III

GENERAL APPROACH

The management of storm water runoff and combined sewer overflows so as not to adversely affect the environment is made extremely difficult due to the highly variable and transient nature of storm water flows and qualities. This is further complicated when the quantity and composition of waters receiving these wastewaters are also time variable, albeit to a lesser extent in most cases.

Conveyance system and storage pond hydraulic capacities that are totally adequate and effective at all times cannot be justified economically, but instead must be designed on the basis of a probability distribution relating capacity with its expected deficiency or inadequacy. Likewise, the hydraulic capacities of wastewater treatment facilities must be limited to values that are expected to be exceeded on occasion. Also the characteristics of the wastewater effluent from these facilities are variable and dependent upon upstream contiguous flow modulation and wastewater treatment performance characteristics. Finally, the real, quantitative effects, both immediate and ultimate, on the receiving waters can only be described by a frequency distribution.

The "steady-state" techniques usually employed to deal with more orderly and less transient sewer systems are grossly inadequate to define, analyze, and evaluate the reactions of a complete integrated water pollution control system for storm water runoff and combined sewer overflows. This program was directed to the development and application of general methods that would be much more responsive to the phenomenological realities of the system and that would permit an assessment from readily available data of the extent of water pollution resulting from various systems designed to control storm water runoff and combined sewer overflows in an urban area.

To provide methods that would have general applicability throughout most of the nation and that at the same time would assure that no feasible solution was overlooked in the analysis, a fairly wide range of system concepts and options were considered and incorporated into a totally integrated system comprised of a collection and conveyance network, a storage reservoir (or none) with associating pumping facilities, a treatment process (or none), and a disposal facility.

Hydraulic capacities in the sewerage components without influent flow regulation were maintained equal. In the cases where a reservoir provided a means for controlled outflow, the subsequent hydraulic capacities reflected an optimized relationship between the reservoir storage capacity and the other downstream system components. Pollutant removals in treatment processes were varied to provide differently performing systems for cost effectiveness evaluations.

For conveyance system design, only the peak instantaneous flow expected to occur during some selected period or recurrence interval is required. The storm water portion of these flows was estimated using a modification of the rational runoff method, which lends itself well to the determination of expected surface runoff quantities from small discrete drainage areas tributary to a single length of sewer. Also the appropriate rainfall intensity-duration-frequency relationships that are required to convert precipitation into surface runoff have been developed and are available in most sections of the country. Peak sanitary sewage flows were estimated using projected populations, per capita wastewater discharge factors, and a capacity factor, which was a function of total contributary population. These peak flows were added to the corresponding peak storm water runoff flows to obtain peak combined sewage design flows.

The determination of storage capacity requirements requires the knowledge of the magnitude and temporal distribution of incoming and outgoing flows. Pond outflow can be controlled to any level and discharge pattern, but a constant outflow probably best meets associated downstream treatment process operating requirements. The determination of storm water runoff quantities as functions of time at storage reservoir locations was accomplished using the unit hydrograph method, which converts rainfall intensity to runoff flow. Extensive historical records of hourly precipitation for most locations in the nation are readily available. The characteristics of the unit hydrograph for particular drainage basins were determined by field measurement and on the basis of available information on the relevant characteristics of the drainage basin. Temporal distribution of sanitary sewage flows was assumed diurnal in nature and was developed from field measurements and other existing data. For estimating hourly combined sewage flows, the diurnal sanitary sewage function was added synchronously to the storm water runoff hydrograph.

Flows to treatment and disposal facilities are dependent upon the nature of contiguous upstream facilities. When a storage pond was provided, the resulting controlled outflow established the hydraulic capacity requirements of the downstream treatment and disposal facilities. Where no flow modulation was provided, the peak flow condition, as determined by the rational method for a given storm recurrence expectancy, established the hydraulic capacity requirements.

Preliminary designs were performed on the various system components for the purpose of estimating construction costs. For the candidate treatment processes, the preliminary designs were based on target objectives for wastewater pollutant removals. These removal rates were then applied to the system performance analysis relating to resulting receiving water pollutant distribution. All facilities costs together with associated operational and maintenance costs were converted to an annual basis for the cost comparison of the many alternatives.

To determine the effect of storm water runoff and combined sewer overflows on receiving waters, a long-term simulated record of storm water flows and concomitant wastewater constituent compositions were developed from hourly precipitation records and mixed synchronously with the corresponding receiving water flows interpolated from historical daily records covering the same period. Concentrations of pollutants in the storm water runoff were estimated on an hourly basis using a model developed in this program that relates the pollutant concentration to the time of pollutant accumulation on the surface of the drainage area and the amount of storm water runoff from the area. Background concentrations of pollutants in the receiving waters were synthesized on an hourly basis from relationships between flow and quality that were established from available measurements and records.

Overall performance of the totally integrated alternative systems was ascertained by the simulation of pollutant removals effected in each system and the subsequent discharge and diminution of the system effluent in the receiving waters. Finally, for the combination of components comprising each complete system, both performance and cost were compared and evaluated within the framework of acceptable receiving water standards.

Section IV

THE STUDY AREA

The City of Sacramento, California, selected as the locale for the study, has both combined and separated sanitary and storm sewers which discharge ultimately to either the American River or the Sacramento River, which bound the study area on two sides. As the Capital of the State of California, the City is experiencing extensive reconstruction and urban redevelopment while retaining large unchanging areas devoted to single-family residences, and thus provides a land-use mix that is believed typical of many communities in the United States with combined sewer systems.

PHYSICAL FEATURES

GEOGRAPHY

The City of Sacramento, California is located at the confluence of the Sacramento and American Rivers in the south-central portion of the broad and fertile Sacramento Valley, a part of the Great Central Valley, which lies between the Sierra Nevada Mountains to the east and the Coast Ranges to the west. San Francisco is approximately 90 miles to the southwest, and Los Angeles is 400 miles to the southeast. The location is shown in Figure 1.

The entire City and a large part of the surrounding County of Sacramento consist of relatively flat topography. Ground elevations within the City range generally between ten and 40 feet, averaging about 25 feet above mean sea level. The foothills of the Sierra Nevada Mountains commence a few miles to the east of the City.

The City of Sacramento lies within the alluvial plain of the Sacramento Valley, formed partly by river erosion and partly by river deposition. Adjacent to the rivers and extending inland for varying distances up to several miles is a belt of recent quarternary alluvium which has been derived by the present stream systems. The remainder of the City, as with a large part of the surrounding region, is covered by the Victor formation, a river flood plain deposit that has been slightly dissected. The debris making up the deposit varies widely, both vertically and laterally, and is heterogeneous, coarse, crudely stratified, and largely unweathered.

The soils within the City consist of those extending inland from the rivers developed on flood plains and recent alluvial fans (infrequent overflow), and those at slightly higher elevations developed on old alluvial plains and terraces. A common characteristic of this latter classification is a hardpan layer at the surface of the substrata, generally

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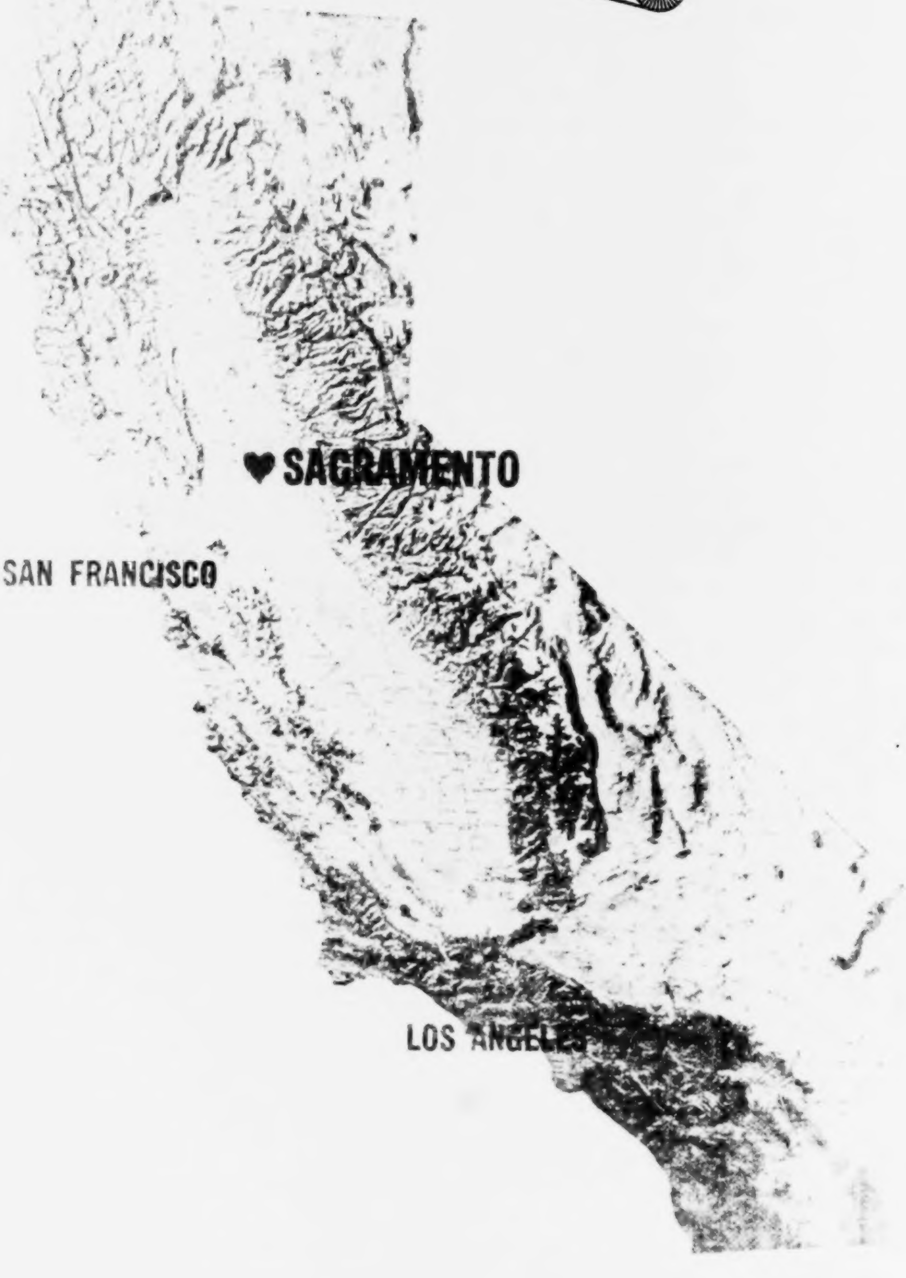

SACRAMENTO
SAN FRANCISCO
LOS ANGELES

Figure 1. LOCATION OF STUDY AREA

from one to four feet below the surface. This layer is a hard, rock-like material varying in thickness up to one foot. The substrata are cemented and hardened, and practically impervious.

HYDROLOGY

The American River flowing from the east bisects the City, and discharges into the Sacramento River which, meandering in a southerly direction, bounds the City on the west.

The Sacramento River is by far the largest stream in California, and is subjected to extensive regulation upstream and on its tributaries. It has an annual runoff past Sacramento of 16,000,000 acre-ft, equivalent to an average flow of 22,000 cfs. Under the California Water Plan, construction work now in progress will divert large quantities of northern California water to southern California. A good portion of this flow will first pass through the reach of the Sacramento River at the City of Sacramento. While no precise quantity can be predicted now, it appears that the minimum flow in the Sacramento River will be on the order of 5,000 cfs during the months of April and May. Flows during August and September would be more nearly 10,000 cfs. In spite of the relatively large degree of upstream regulation, there will continue to be large flood flows passing Sacramento, estimated at approximately 50,000 cfs as an average peak flow.

The American River is one of the larger streams in California with an average annual runoff of 2,700,000 acre-ft. It is regulated some 25 miles above Sacramento by Folsom Dam and Reservoir. Additional reservoir projects located upstream are planned. Under a program soon to be implemented, most of the American River runoff will be diverted for irrigation and other purposes at Nimbus, a few miles to the east of Sacramento. Under ultimate development at the American River, it is planned that only sufficient water will be released past the Nimbus Diversion Dam to provide the water supply for Sacramento and adjacent communities and to preserve the fisheries resource. The minimum quantity to reach the Sacramento River will thus be only 250 cfs during certain months of the year.

CLIMATE

The Sacramento Valley enjoys a mild climate with an abundance of sunshine. Cloudless skies prevail during the summer and during much of the spring and autumn. Mountains surround the Valley to the west, north, and east. Because of the shielding influence of the mountains, heavy rainfall and excessive winds are rare.

Average annual rainfall within the City is close to 17 inches. Practically all of this rainfall occurs during the period from November through April, yet rain in measurable amounts occurs only on about ten days each month during this rainy season.

Summers are hot and dry with average maximum temperatures of about 90° F and occasional extremes as high as 114° F; nights are generally cool. Most autumn and spring days are cloudless. Winters are of moderate intensity, although below-freezing temperatures occur occasionally.

Relative humidity ranges from about 60 to 90 percent throughout the day during the winter months, and about 30 percent during the summer. The low diurnal reading is generally noted near mid-afternoon.

The extremely low relative humidity that accompanies high temperatures during the daytime in the summer months should be considered when comparing temperatures of Sacramento with those of other cities in more humid regions. Thunderstorms are few in number and usually mild in character. Snow falls so rarely and in such small amounts that its occurrence may be disregarded as a climatic feature. Heavy fog occurs mostly in mid-winter, never in summer, and seldom in spring or autumn. Light and moderate fogs are most frequent, and may come anytime during the wet, cold season. The fog is usually the radiational cooling type and is confined to the early morning hours. An occasional winter fog, under stagnant atmospheric conditions, may persist for several days.

Table 1 contains a summary of important climatological data for the area, which have been compiled in Sacramento since 1849.

Table 1

MEAN CLIMATOLOGICAL DATA FOR SACRAMENTO, CALIFORNIA

Month	Rainfall, in.	Temperature, °F	Maximum Temperature, °F	Minimum Temperature, °F	Relative Humidity at 4 p. m.	Percent of Possible Sunshine	Prevailing Wind Direction
Jan	3.2	45	53	37	73	44	SE
Feb	3.0	49	59	40	62	60	SSE
Mar	2.4	53	65	42	53	69	SW
Apr	1.4	58	71	45	45	79	SW
May	0.6	64	78	50	42	83	SW
Jun	0.1	70	87	54	31	91	SW
Jul	0	75	93	57	27	98	SSW
Aug	0	74	92	56	28	95	SW
Sep	0.2	72	88	55	33	93	SW
Oct	0.8	64	78	49	45	85	SW
Nov	1.4	53	64	42	62	64	NNW
Dec	3.2	46	55	38	80	44	SSW
Yr	16.3	60	74	47	48	78	SW

CITY EVOLUTION

EARLY YEARS

Sacramento had its original beginning in 1839 when Captain Sutter built his fort at what is now 27th and K Streets and established an embarcadero on the waterfront of the Sacramento River just below its confluence with the American River. When gold was discovered at Sutter's Mill in nearby Coloma in 1848, Sacramento became a boom town. By July of 1849, there were approximately 100 buildings along the waterfront area now called Old Sacramento.

As shown by Figure 2, which presents the map of the City dated October 1859, the gridiron street pattern which today characterizes that portion of Sacramento called the Old City was already established. Also in evidence on the 1859 map were vast areas of the City that were subjected to periodic flooding from storm overflows of both the Sacramento and American Rivers.

Because of severe flooding in 1861 and 1862, the street levels near the waterfront were raised one story in building height in 1864, accomplished simply by filling the original ground floor to the second level. The attempt to protect against the flooding which plagued the City in the early years is still much in evidence in the architecture of older homes with the main floor level elevated approximately one story above the adjacent ground. Even today, very few residences contain basements. The flood threat has gradually been alleviated by the construction of an extensive levee system along the Rivers. In more recent times, the Rivers themselves have come under increasing control through numerous water conservation and flood control projects throughout their upper reaches.

The original boundaries of the City enclosing a 4.5-square mile area remained more or less intact until 1911 when a 9.5-square mile annexation increased the total area to 14 square miles. Starting in 1946, the City has expanded areawise through many annexations, until today it encompasses 94 square miles of territory, some of which is yet undeveloped and devoted to agricultural purposes, as shown in Figure 3.

RECENT YEARS

Characteristic of many other cities at comparable stages of their development, the older portion of Sacramento between the riverfront and the Capital to the east experienced an undesirable change starting in the late 1920's. The strong relationship between river traffic, railroads, industry, and business no longer provided a common bond and the area was abandoned to the forces of neglect and changed land use. With the business center moving eastward away from the deteriorating core, the

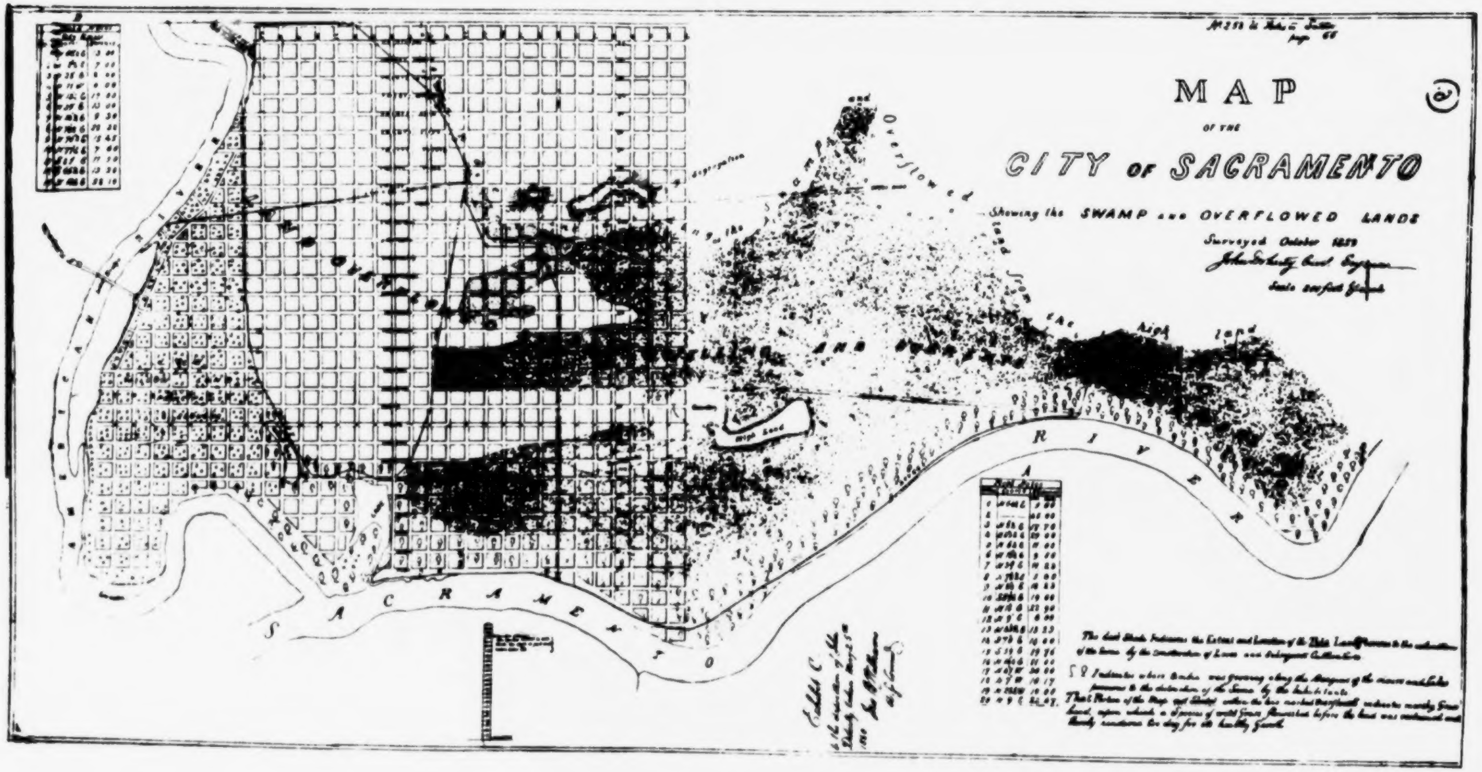

MAP
OF THE
CITY OF SACRAMENTO
Showing the SWAMP and OVERFLOWED LANDS
Surveyed October 1859
Overflowed
High land
SACRAMENTO
RIVER
AMERICAN RIVER

Figure 2. MAP OF THE CITY OF SACRAMENTO - 1859

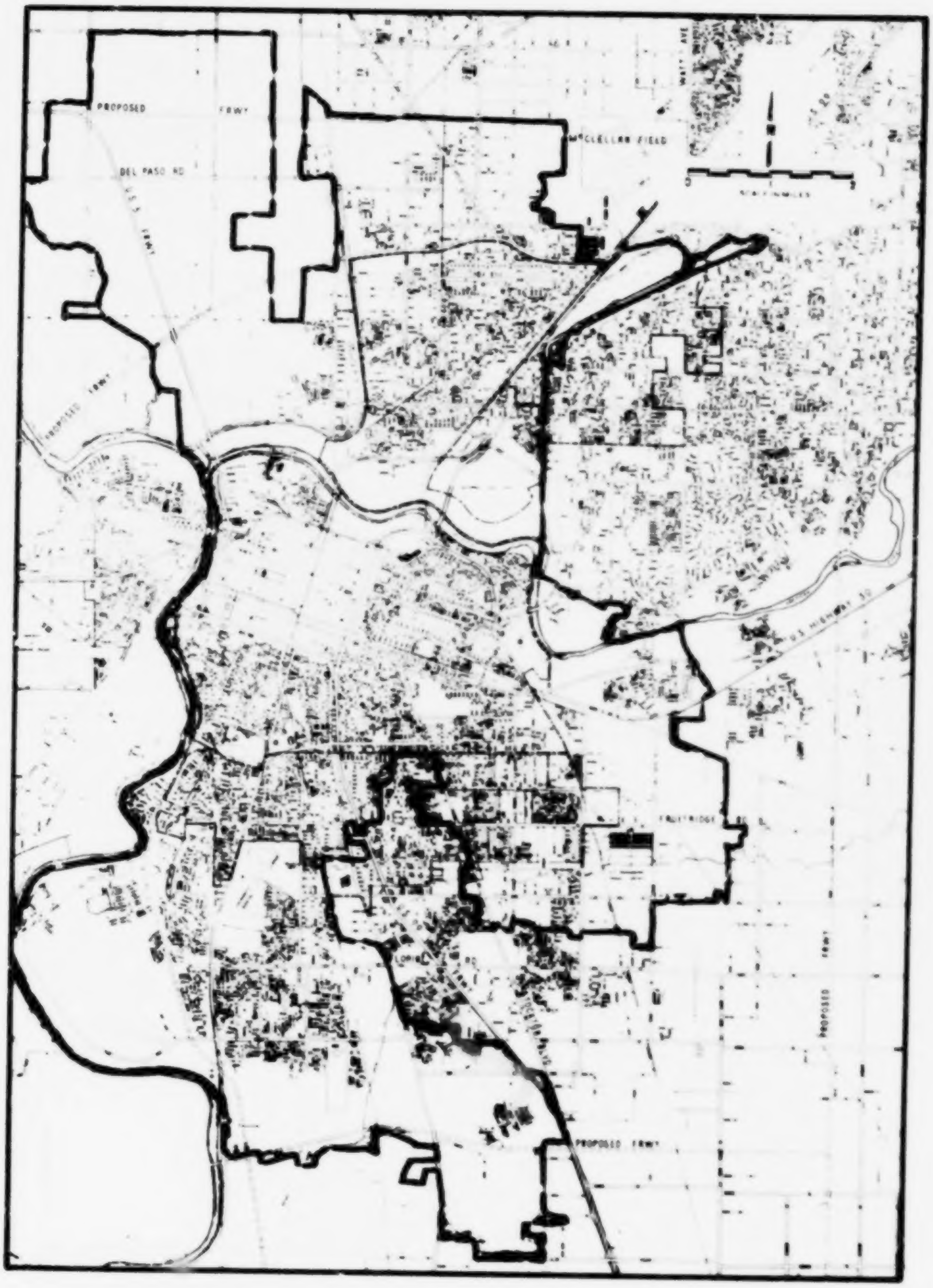

PROPOSED FRWY
DEL PASO RD
McCLELLAN FIELD
PROPOSED FRWY
U.S. HIGHWAY 50
FRUITRIDGE
PROPOSED FRWY
PROPOSED FRWY

Figure 3. SACRAMENTO CITY BOUNDARY - 1969

blight was gradually infecting a larger and larger area of the City.

Recognizing the need for corrective action, the City Council in 1948 authorized a survey of the deteriorated area, which led subsequently to an allocation of federal funds and the activation of the Sacramento Redevelopment Agency in 1950, pursuant to the provisions of the California Community Redevelopment Law of 1945. The results achieved through urban renewal to date are far-reaching. Much of the blighted area with attendant problems has been cleared. Already in evidence is a well-planned mix of new commercial facilities, with more to come, as shown in Figure 4. As a matter of interest, the historic Old Sacramento or Embarcadero area long the waterfront will be restored or reconstructed to capture again the exciting era of the bustling frontier city.

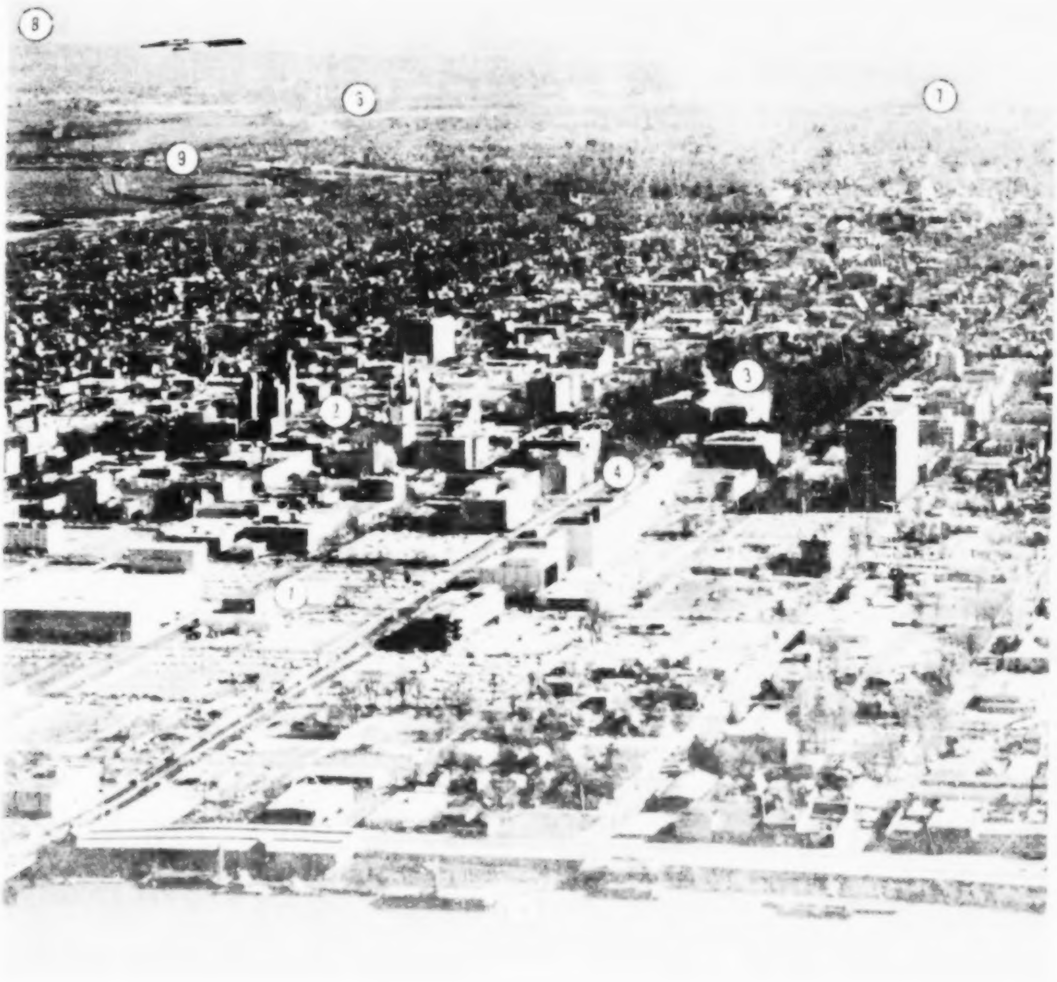
Somewhat concurrently with the work of the Redevelopment Agency, the State Legislature in 1959 created the Capital Building and Planning Commission to provide for the orderly development of future State buildings in the City of Sacramento. A master plan has been developed with a stated purpose to give California a noble and monumental seat of government. The comprehensive plan projects requirements for an orderly growth of facilities to accommodate future State employees. The State is acquiring land on all sides of the Capital, excluding the commercial shipping area to the north, in order to meet its future needs. Modern high-rise office structures are already in evidence and portend the dramatic changes yet to come.

Sacramento State College is located along the American River in the eastern section of the City. The College enrollment in 1969 was close to 10,000 students, and is expected to double in the next 12 to 14 years.

Even as late as 1950, the majority of the Sacramento metropolitan area population resided within the boundary of the City. Since 1950, however, an explosive population growth has occurred in the area surrounding the City, primarily to the northeast. The sharp increase in growth rate of the metropolitan area during the 1950's was caused in part by the expansion of two major employers, McClellan Air Force Base and Aerojet-General Corporation. With other industrial companies also selecting Sacramento for the site of new plants, the area's economy which previously had been based primarily on government and agriculture now included a significant industrial component.

Sacramento is the county seat of Sacramento County. The estimated population of Sacramento County in 1968 was 670,000 of which approximately 275,000 resided within the City of Sacramento.

The City of Sacramento and the surrounding metropolitan region is expected to continue as an important and growing commercial and industrial center. The recent development of the Sacramento-Yolo Port which accommodates oceanic vessels will enhance the area as a desirable location for new industry. Along with the benefits to be derived from new



1. Sacramento Redevelopment Area
2. Sacramento Central Business District
3. California State Capitol and Complex
4. Federal Buildings
5. Sacramento River
6. American River
7. Sacramento State College
8. Lake Folsom
9. California State Fair (new site)

Figure 4. AERIAL PHOTO OF CENTRAL SACRAMENTO LOOKING EAST

industrial development, the economy of Sacramento will continue to be based strongly on government and agriculture, both of which are expected to expand in future years.

GOVERNMENT

In 1921, the City adopted a City Charter which provides a 9-member elected City Council. The members of the City Council are elected by the city-at-large to 2-year terms. There are no other elected City officials. By custom, the councilman receiving the most votes in the biennial election is named Mayor by the Council. The City functions under a Council-Manager form of government with the City Manager responsible to the City Council.

FINANCIAL

The 1968 total assessed valuation within the City was $525 million, down slightly in the past several years because of extensive land clearing for new freeway construction and urban renewal. Historically, assessed valuation represents about 25 percent of market value. The 1968 City tax rate was $2.17 per $100.00 of assessed valuation, which together with other diversified revenue sources provides funds for governmental services. Water and sewer utilities operated by the City charge rates which are adequate both to support operations and provide capital development funds, including debt service costs for these utilities.

Currently the City budget totals about $35 million which includes those activities such as the Department of Water and Sewers which functions on a self-supporting basis. Debt obligations as of August 1968 totalled $113 million, which reduced to $88 million after subtracting all self-supporting bonds. The $88 million bonded debt represents about 17 percent of assessed valuation.

THE COMBINED SEWER AREA

RELATION TO CITY

The primary study area of this program is by definition that area covered by the combined sewer system in Sacramento. However, many aspects of the combined sewer investigation extend beyond this specific area into other parts of the City and metropolitan region. The primary study area is further defined as that part of the overall combined sewer system that collects and conveys municipal sewage via the combined system to a common location.

While also a part of the total system, the municipal collection districts north of the American River which feed into the combined system are not considered a part of the primary study area. Storm water separation projects in the upper reaches of the combined sewer system are considered within the primary study area, since sanitary sewage continues to be conveyed by the older combined sewers, which downstream remain truly combined. In this report, the primary study area henceforth will be called the Study Area, the separated storm water part of the Study Area will be called the Separated Area, and the truly combined part of the Study Area will be called the Combined Area.

The Study Area, shown in Figure 5, covers 18.8 square miles of the older or central portion of the City, commonly referred to as the Main City Section. For many years the Study Area comprised the entire City plus what was then considered to be suburban countryside. The Study Area is, therefore, the locale for most of the activities relating to city growth and characterization as described previously.

POPULATION

With its general location in the older developed portion of Sacramento, the Study Area has in recent years experienced, and will continue to experience, a slower growth rate in residential population than the less-developed surrounding regions. On the other hand, primarily because of the impact of State government and Sacramento State College growth, the work force or employee population within the Study Area is expected to grow at a much faster rate in the years to come than the corresponding residential population.

Because of the unusually high ratio of employees to residents, it was deemed advisable to consider the employee population separately. An employee is considered as one who is either regularly employed in the Study Area or for other reasons spends approximately eight hours each day within the Study Area away from home. Thus, a tourist, a shopper, or a student might contribute to the employee population. The employee may or may not also be a resident of the Study Area.

From estimates prepared by the City Planning Department, the July 1968 residential population of the Study Area was 111,395. Based on data collected from several sources, the July 1968 worker population of the Study Area was estimated at 93,913. The above populations do not include the areas to the north of the American River which contribute sanitary sewage to the combined sewer system.

LAND USE

The structure of the Study Area from a land-use standpoint is typical of many other American cities in comparable stages of their evolutionary development. As the City grew, changing patterns in land use developed which, for varying reasons, resulted in sub-standard residential housing near the central core. Fortunately for Sacramento, the impetus of urban renewal plus the initiation of other long-range planning programs have served to arrest this undesirable trend in much of the older areas.

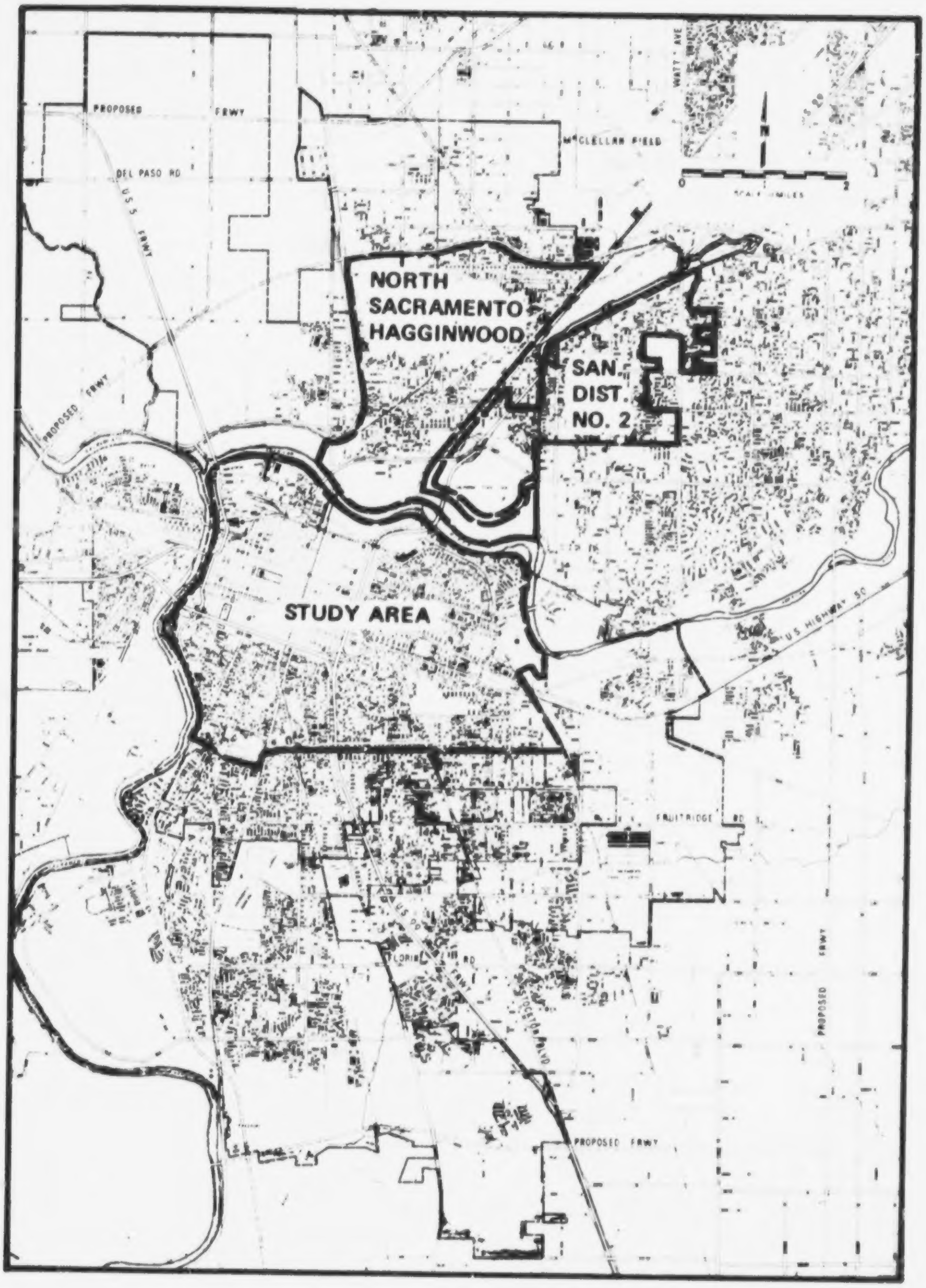

PROPOSED FRWY
DEL PASO RD
U.S. FRWY
McCLELLAN FIELD
WATT AVE
N
0
1
2
SCALE-MILES
NORTH SACRAMENTO HAGGINWOOD
SAN. DIST. NO. 2
PROPOSED FRWY
STUDY AREA
U.S. HIGHWAY 50
FRUITRIDGE RD
FLORIN RD
STOCKTON BLVD
PROPOSED FRWY
PROPOSED FRWY

Figure 5. SACRAMENTO COMBINED SEWER SYSTEM

The tremendous growth of the Sacramento metropolitan region during the 1950's has already been mentioned. This surge of population into surrounding suburbs is, of course, a phenomenon not unique with Sacramento. As elsewhere in the country, the conversion of open land to mass housing developments, shopping centers, and other attendant facilities could not help but alter the orientation of commercial and business activities within the City.

While a few pockets of undeveloped land still exist, the Study Area today may be characterized as essentially built-up. The aerial photograph covering the entire Study Area and adjacent regions presented in Figure 6 visually depicts this condition. Future residential growth will be derived primarily by transition from single-family to multiple-family housing. Future worker growth will be accommodated by displacing present residential and light commercial areas with construction of high-rise facilities. The commercial and nongovernmental business activity within the Study Area is expected to grow moderately along with the residential population. Likewise, because of available land constraints, industrial activity within the Study Area will experience only moderate growth. Total utilization of available land and a much more intensive land use is projected for the future. Rates of growth will be increasingly constrained with time as the Study Area gradually approaches saturation.

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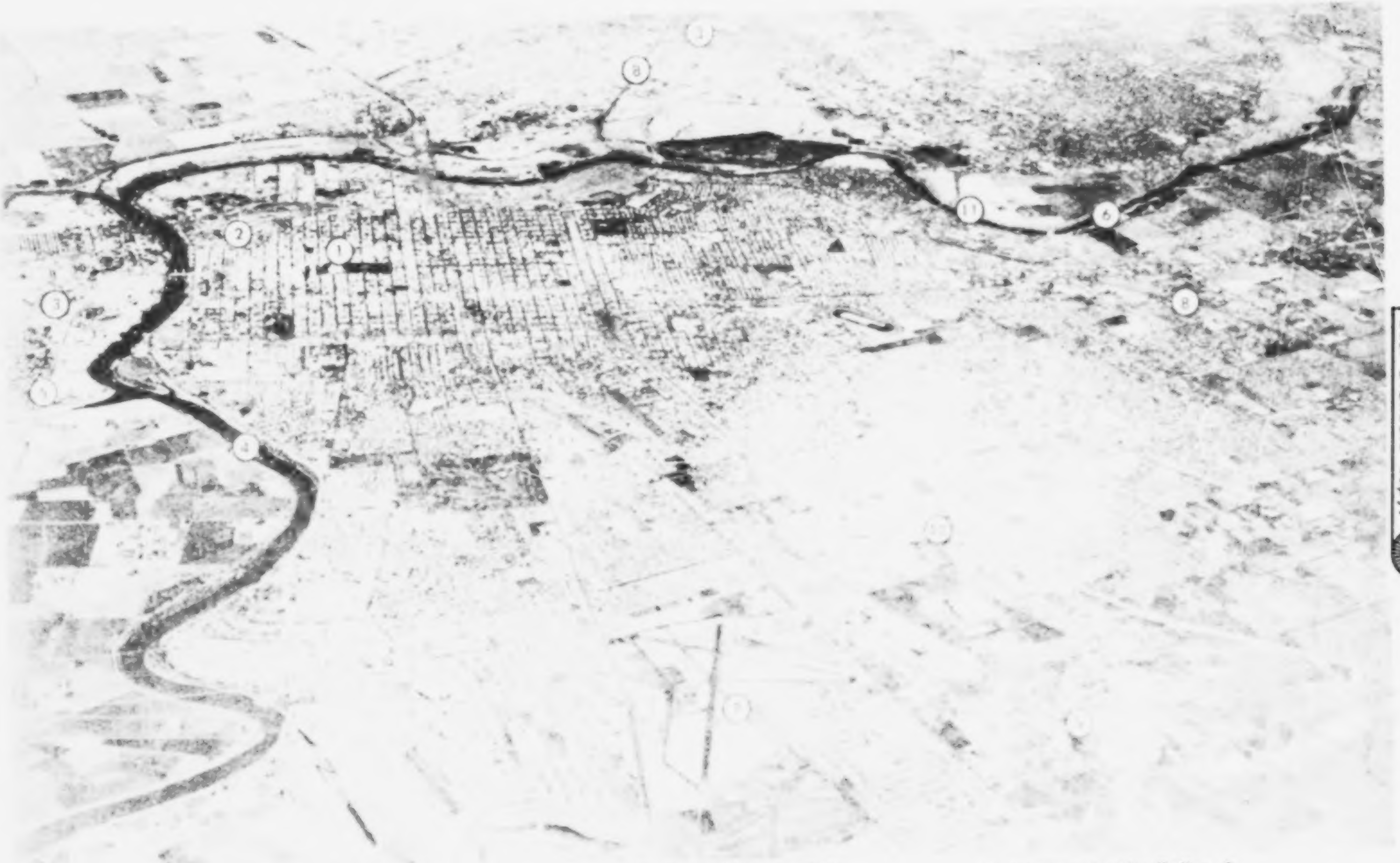

1. California State Capitol
2. Redevelopment and Central Business District
3. U. S. 40 - Interstate 80
4. Sacramento River
5. Port of Sacramento
6. American River
7. Sacramento Municipal Airport
8. Southern Pacific Railroad
9. Western Pacific Railroad
10. U. S. 50-99
11. Sacramento State College

Figure 6. AERIAL VIEW OF STUDY AREA AND SURROUNDING REGION

Section V

THE WASTEWATER SYSTEM

There are over 30 organizations within Sacramento County providing sewerage services, representing for the most part incorporated cities, county sanitary districts, and local maintenance districts. Approximately 75 percent of the 670,000 population of the County in 1968 was served by these community-type sewer systems. This percentage is quite high and is attributable to the progressive attitude of the County Department of Public Works in creating the necessary sewer districts to keep pace with the rapid growth.

Individual sewage disposal facilities are required in the areas not served by community sewer systems. The County Health Department has cognizance over individual systems that includes approval of plans before issuance of building permits. County zoning ordinances require new residential developments to maintain larger minimum lot sizes in order to quality for approval of individual septic-tank and leaching-field systems. This requirement favors the development of public sewer systems where higher density land use is evolving.

Nearly all of the sewage authorities within the County provide primary or secondary treatment to waste flows which are discharged typically either to small streams which flow to the American or Sacramento Rivers or directly to the Rivers.

Since the Sacramento and American Rivers have huge watersheds extending above Sacramento County, a considerable amount of sewage is discharged to these Rivers before reaching Sacramento. The waste products associated with agricultural activity in the Sacramento Valley also contribute to the pollution of the Rivers. The effects of these remote discharges at Sacramento are ameliorated due to the dilution provided and the natural purification processes taking place in the streams.

Nearby, but outside of the County, the West Sacramento Sewer District operates a sewer system and treatment plant immediately across the Sacramento River and adjacent to the Study Area. The treatment plant provides primary treatment.

The City of Roseville, located just to the north of the County boundary, operates a sewer system with treatment plant that discharges via a creek which flows to the American River through the north central portion of Sacramento County. The major thrust of future urban growth in the area appears to be toward Roseville from Sacramento.

The one other major waste contributor within the general area is the American Crystal Sugar Company located in Clarksburg on the west bank of the Sacramento River and south of the City. The sugar processing operation is seasonal in nature and the liquid waste receives treatment by settling tanks and holding ponds with effluent discharging to the Sacramento River.

FACILITIES DESCRIPTION

The City's overall sewer service area contains a system of storm and sanitary collection sewers, three major pumping installations, 29 sanitary and 67 storm water lift stations, and two treatment plants with appurtenant outfall facilities. The sewers in the City comprise a system approximately 900 miles in length and range in size from 6-in. sanitary lines to 114-in. combined sewage trunk drains.

Within the City of Sacramento service area, the City Engineer, reporting to the City Manager is responsible for the City's municipal water supply and wastewater systems, which are operated by the Division of Water and Sewers. The Division has a staff of approximately 230 employees including engineers, technicians, clerks, and operating and maintenance personnel.

The City's sewer service area, shown in Figure 7, in 1968 encompassed 58 square miles, with an estimated population of 279,000. Of this area, six square miles containing 38,000 persons lies outside the City limits. The total area of the City is 94 square miles. The portion of Sacramento not included in the City's sewer service area is provided with sewage collection and disposal by five different systems, three of which have their own treatment plants. These systems were installed by County sanitation districts in areas subsequently annexed by the City. The systems continue to impose their own tax rates, connection charges, and service charges and are operated under the control and supervision of the County Department of Public Works. County Sanitation Districts Nos. 1 and 2, which have contractual arrangements with the City for sewage treatment, serve areas both inside and outside the City limits and account for the only service of this type provided by the City for nonresidents.

Prior to about 1948, which marked the beginning of an extensive growth through annexation, the City's sewer service area consisted essentially of the area covered by the combined sewer system and is the Study Area of this program. The combined sewage flows by gravity to a common point, Sump No. 2, near the southwest corner of the Study Area, and is pumped to the Sacramento Main Treatment Plant located in the southwest section of the City on a 35-acre site. The Main Treatment Plant, which accepts sewage from several other service areas, was completed in 1954 at a cost of $4.4 million with a nominal design capacity of 76 mgd. This plant, which soon will incorporate secondary treatment, provides primary treatment before discharging disinfected effluent through an outfall line to the Sacramento River.

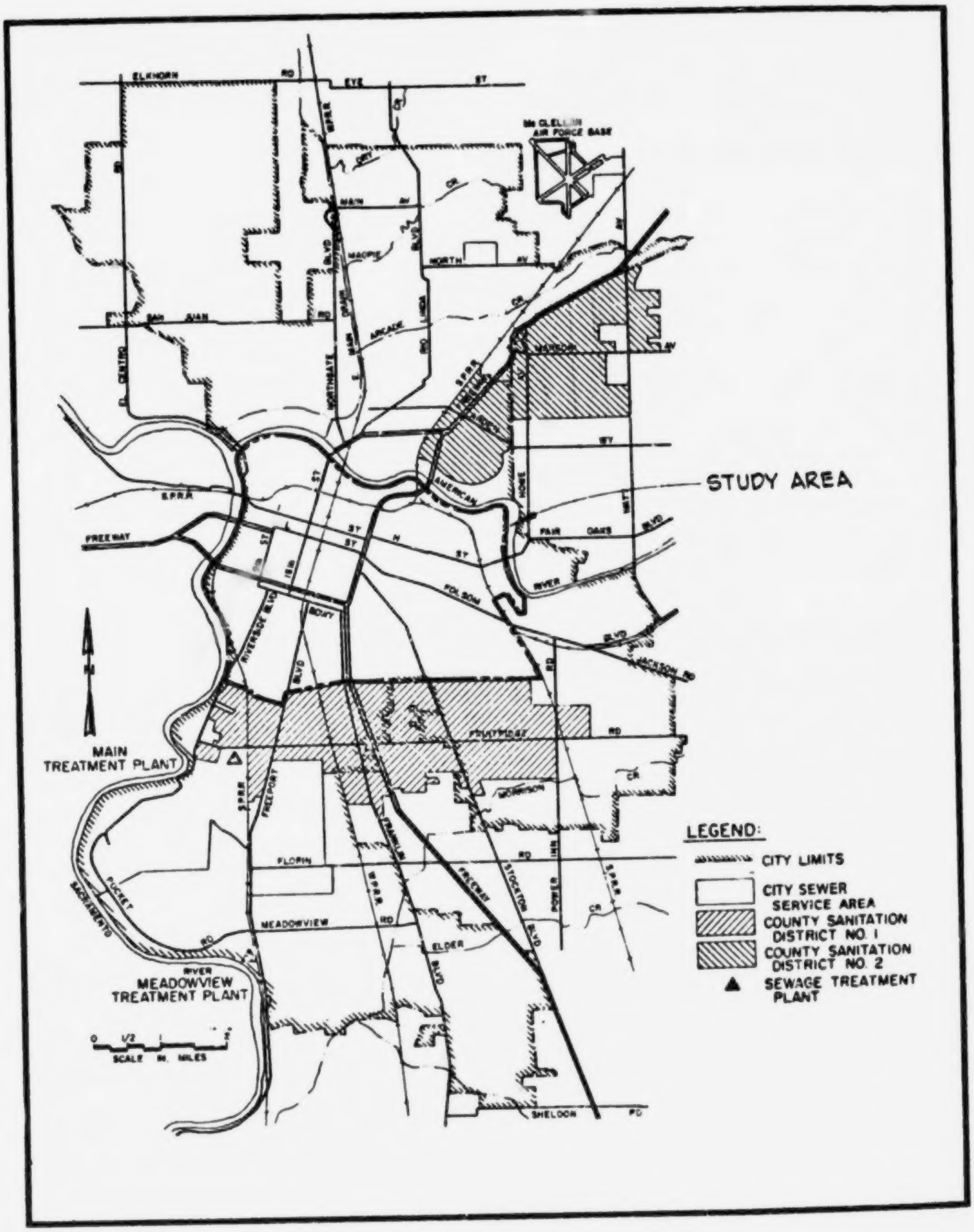
ELKHORN RD
EYE ST
McCLELLAN AIR FORCE BASE
NORTH AV
ARCADE
S.P.R.R.
AMERICAN RIVER
STUDY AREA
FREEWAY
FAIR OAKS BLVD
FOLSOM BLVD
JACKSON RD
FRUITRIDGE RD
MORRISON CR
MAIN TREATMENT PLANT
FREEPORT BLVD
FLORIN RD
MEADOWVIEW RD
ELDER
STOCKTON BLVD
SACRAMENTO RIVER
MEADOWVIEW TREATMENT PLANT
SHELDON RD
0 1/2 1
SCALE IN MILES
N
LEGEND:
CITY LIMITS
CITY SEWER SERVICE AREA
COUNTY SANITATION DISTRICT NO. 1
COUNTY SANITATION DISTRICT NO. 2
SEWAGE TREATMENT PLANT

Figure 7. WASTEWATER TREATMENT SERVICE AREAS IN CITY OF SACRAMENTO

The City also owns and operates the Meadowview Treatment Plant located near the southern City limits along the Sacramento River. This 2.6-mgd plant, acquired by the City through annexation, provides primary treatment and disinfection.

SEWERAGE SYSTEM DEVELOPMENT

Design drawings prepared for construction of segments of the existing combined sewer system are on file in the Sacramento City Engineering Department which date back to circa 1880. Some portions of the system are undoubtedly older. The construction and maintenance of sewers and storm drains have been a City utility operation since 1878.

In common with many cities throughout the country, the combining of sanitary wastes and storm water into a single sewer system was adopted as a basic policy in Sacramento for all sewer system construction over a time span of many years. The combined sewer system functioned adequately and fulfilled its intended purpose in meeting the minimum needs of the City, particularly during the period when the population was only a fraction of that served today and treatment of sewage prior to river disposal was not considered mandatory.

Because of extensions to the combined sewer system, certain portions of the larger trunk sewers experienced severe overloading from time to time, which was relieved by construction of bypass sewers to other trunk lines. Over a span of several decades, these cross-connections were added to the system together with various types of diversion weirs.

The combined sewer system reached its maximum areal coverage in the late 1940's. By that time, the extension outward had in some instances reached the boundary of other sewerage districts and elsewhere the decision was made not to further burden the system with additional storm water flows. Also, in keeping with the awareness of the times on the need for improvement, efforts were undertaken to provide treatment facilities for sanitary sewage. No combined sewers have been constructed in Sacramento subsequent to 1946.

During the early years of the system, all sewage flowed to a common pumping station, and thence discharged directly to the Sacramento River. The original pumping station was replaced in 1908 by the existing Sump No. 1, which is shown on Figure 8. As the City grew to the east and south, trunk lines were constructed to a new pumping station, the existing Sump No. 2, built in 1916. Both pumping stations have been expanded from time to time. Until 1954 when Sacramento completed its first wastewater treatment facility, all flows were discharged untreated directly into the Sacramento River.

In 1954, a 72-in. force main was placed in operation from Sump No. 2 to the Sacramento Main Treatment Plant. From that time, all flow bypasses Sump No. 1 except during storms and is pumped via the force

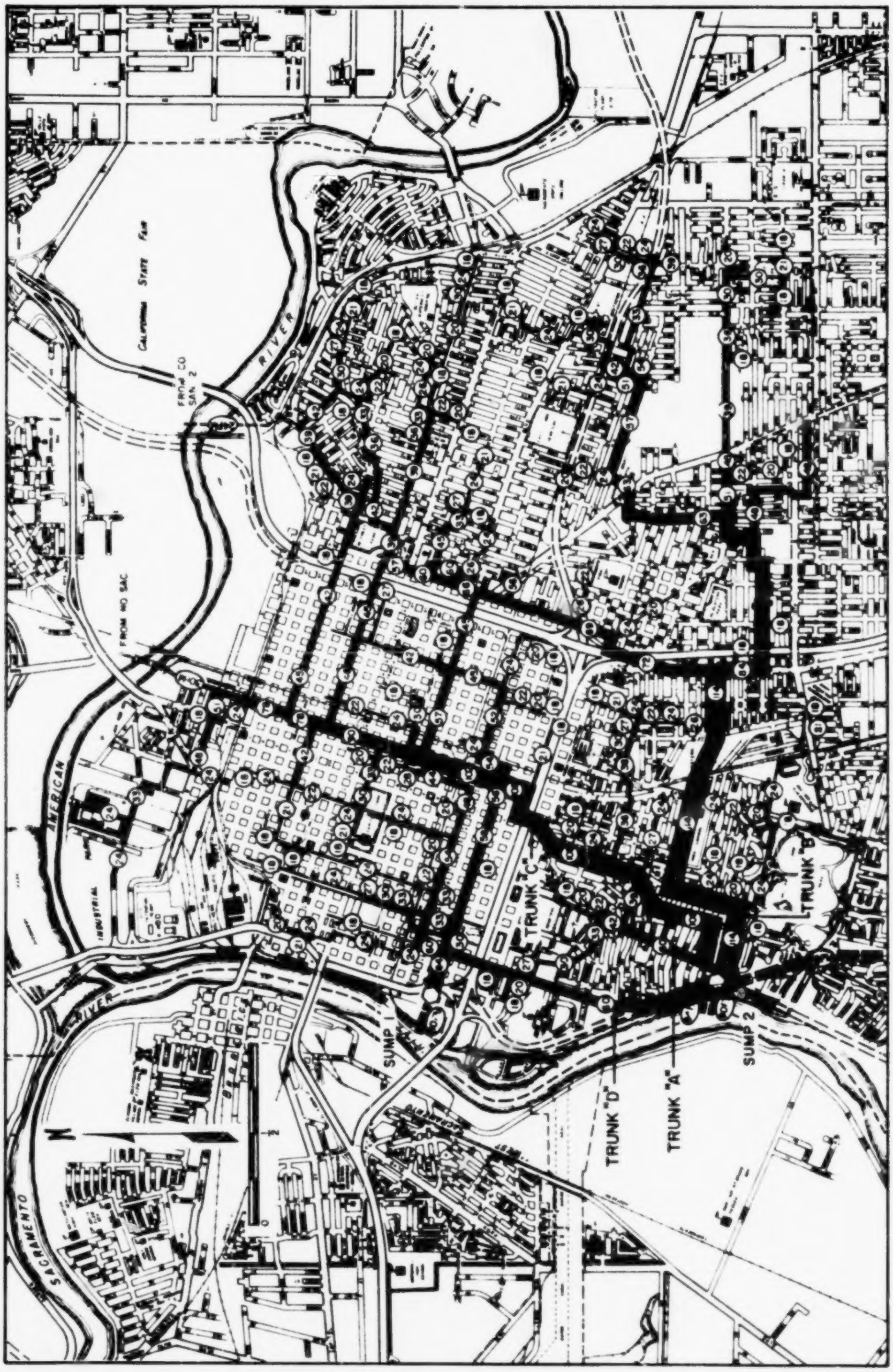
SACRAMENTO RIVER
AMERICAN RIVER
CALIFORNIA STATE FAIR
FROM CO SAN 2
FROM NO SAC
INDUSTRIAL
SUMP 1
SUMP 2
TRUNK "A"
TRUNK "B"
TRUNK "C"
TRUNK "D"
N

Figure 8. EXISTING COMBINED SEWER SYSTEM

main from Sump No. 2 to the treatment plant. Depending upon the severity of a particular storm, the combined flows are diverted either totally or partially to the Sacramento River directly from Sump No. 2, with Sump No. 1 also utilized as needed for additional flow diversion directly to the River.

Three separate sanitation districts north of the American River and outside the City previously discharged sanitary sewage to the combined sewer system. The area encompassed by two of these districts, North Sacramento and Hagginwood, has since been annexed by the City and a single force main now discharges into the upper reaches of the combined sewer system and continues via gravity flow to Sump No. 2. County Sanitation District No. 2, located partially within the City, also discharges sanitary flow through a force main under the American River to the combined sewer system in a similar manner.

Experience from system overloading and attendant flooding gave cause for the City to undertake measures to update the combined sewer system and relieve the chronic flooding which persisted in certain areas, even from storms of moderate intensity. The earlier method of relieving the system at critical points by the construction of cross-connections between main trunks was no longer considered feasible. In fact, with the passage of time, the rationale used in the selection of cross-connection routings was no longer in clear evidence in some instances.

Starting in the early 1960's the City undertook a new approach in the modernization of the combined sewer system. An improvement program was initiated wherein new separated storm sewers were constructed in selected areas, with the existing combined sewer system continuing to be utilized for sanitary flows only. There were several reasons for selecting the new separate system for storm water runoff rather than sanitary sewage. First the areas selected were located conveniently near discharge points to the Rivers. Second, the storm flows would not, by standards heretofore acceptable, require additional treatment facilities. Third, the disruption to existing sewer facilities would be considerably less because residential sewer connections would not require disconnection and reestablishment. Finally, the basic problem of relieving the combined sewers during storms would best be served with a modern adequately designed storm sewer system.

After completion of two of the separation projects, the residents of Sacramento in 1964 voted approval for the sale of $15 million in bonds for continuation of this improvement work. Subsequently, three other separation projects have been implemented under the bond program. In addition, one other separation project has recently been completed in the industrial district along the American River, financed through formation of a local sewer assessment district.

COLLECTION AND CONVEYANCE

The Study Area covers 10,772 acres which is served by the combined sewer network (see Figure 8). Of this total area, seven separated storm water sewerage systems cover 2,881 acres. Also, there are two parcels of land--308 acres at Sacramento State College and 545 acres along the American River adjacent to the Elvas Freeway--that presently are not served by any type of storm sewer system. There remains an area of 7,038 acres where the system continues to function as a truly combined sewer system, receiving both sanitary wastes and storm water drainage.

The portions of the Study Area where the seven separated storm water systems have been superimposed over the combined sewer system to form the Separated Area are shown in Figure 9. Typically, each storm water system drains to a lift station, identified in Figure 9, from whence the runoff is discharged to one of the Rivers through a short force main. Treatment of separated storm water is not provided.

At present, the storm water conveyance system identified by Sump A in Figure 9 discharges back into the combined sewer system. This was ignored in this study on the assumption that the separated storm flow will eventually be accommodated through adjacent facilities to the south in County Sanitation District No. 1.

There are no immediate plans in progress for construction of new separate storm systems within the Study Area. Meanwhile, overloading of the combined sewer system continues to occur, albeit to a lesser degree than experienced prior to initiation of the improvement program.

TREATMENT

Except when combined sewer overflows necessitate direct diversion to the Sacramento River, all sanitary flow from the combined sewer system is pumped approximately two miles to the Main Treatment Plant from Sump No. 2 through a 72-in. force main. The maximum peak discharge to the treatment plant is about 86 mgd. In addition to the flow from Sump No. 2, the Main Treatment Plant receives sanitary sewage from County Sanitation District No. 1. In recent years, it has been the practice during moderate storms to split the combined flow from Sump No. 2 between the Main Treatment Plant and the River. For larger runoffs, all combined flow is bypassed to the Sacramento River.

Planning is currently in progress for construction of biofilter secondary sewage treatment facilities at the Main Treatment Plant which would improve effluent quality in line with higher anticipated receiving water quality standards. The improvement program will also include increasing primary treatment capacity from a nominal 76 to 95 mgd.

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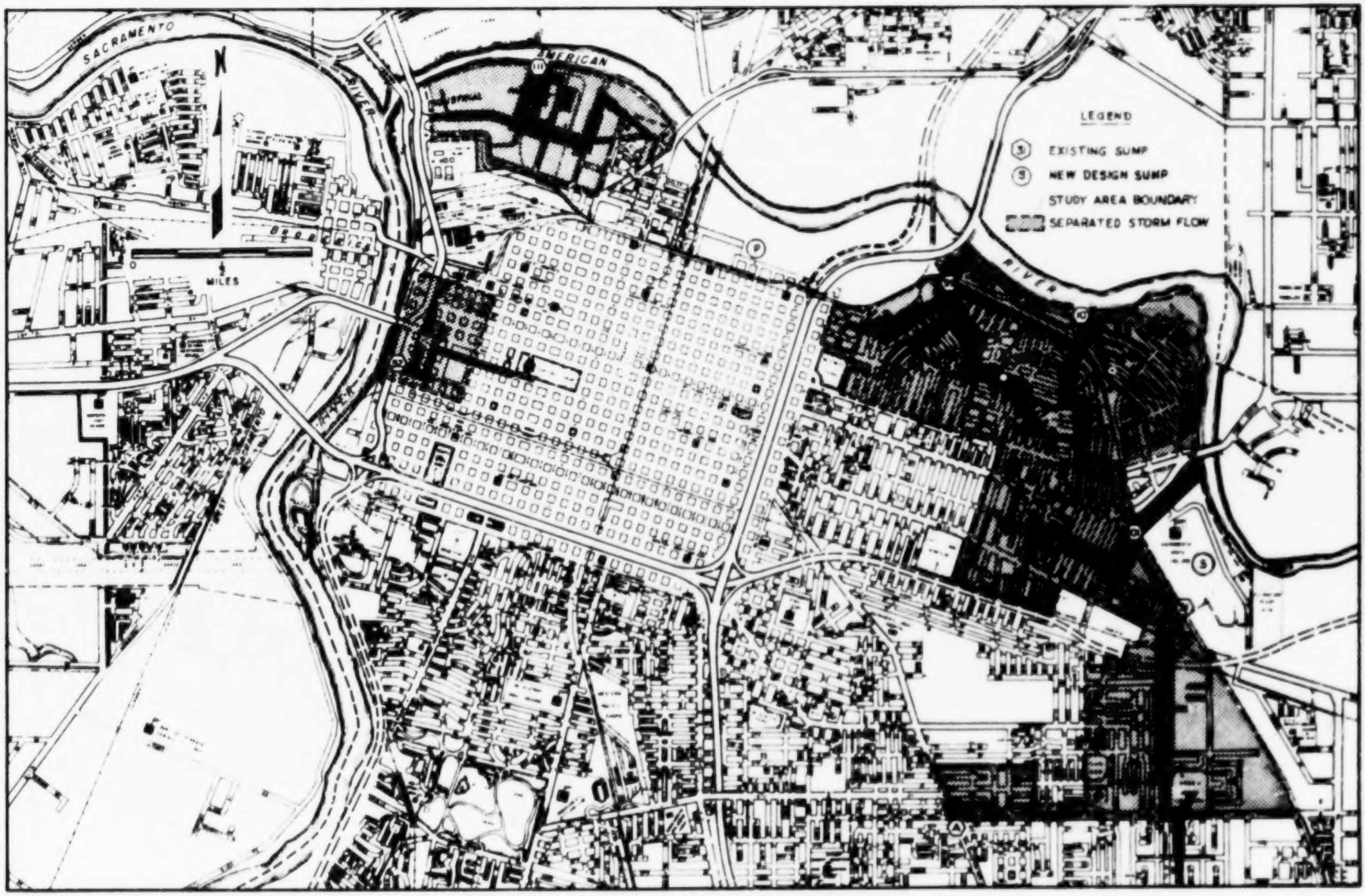
LEGEND
Ⓢ EXISTING SUMP
Ⓢ NEW DESIGN SUMP
STUDY AREA BOUNDARY
▨ SEPARATED STORM FLOW
SACRAMENTO RIVER
AMERICAN RIVER
N
0
½
MILES

Figure 9. EXISTING SEPARATED STORM WATER SEWER SYSTEMS

MAIN SUMPS

Sump No. 2 contains eleven pumps arranged as shown in Figure 10. Because of the configuration of incoming trunks and the baffling of sump wells, only Pumps No. 5 through 8 are used for normal sanitary flows, with Pumps No. 1 through 4 used when flow depth in the well exceeds 3.5 feet. Pumps No. 1 through 8 can be utilized either for flows to the treatment plant or to the River, with flow quantities adjusted by selection of pump or combination of pumps of varying capacities. Pumps No. 9 through 11 are used only for diversion of combined flows to the Sacramento River.

The Sump No. 2 pumps are protected by manually cleaned vertical bar screens. Screenings are hauled away by truck. No flow metering or chlorination facilities are provided. All pumps have electrical motor drives energized from a single transformer station. The pump station is manually controlled by visually gaging incoming sewage levels and selecting pumps as required. The necessity for bypassing to the River during a storm is determined by the operator. A continuous 24-hour watch at the station is required.

At present, all flow normally bypasses Sump No. 1 and flows by gravity through a 60-in. trunk to Sump No. 2. However, during periods of excessive rainfall, the inflow is pumped from Sump No. 1 directly into the Sacramento River through a 60-in. outfall. As shown in Figure 11, there are six pumps at the station, with a total capacity of 240 mgd. Three of the pumps are electrical motor driven and three are gas engine driven.

Sump No. 1 is normally unattended. The three electrical motor driven pumps may be controlled by the operator at Sump No. 2, but an attendant is required at the station to operate the gas engine driven pumps. The cleaning of bar screens also requires an attendant during periods of pumping.

WASTEWATER AND RAINFALL CHARACTERISTICS

Information on the characteristics of untreated combined sewage and separated wastewater runoff during dry weather and wet weather was obtained through a program of field sampling and laboratory analysis. Due to the timing of program commencement in relation to the wet-weather season in Sacramento, and the need to establish certain wastewater characteristics early in the study, estimated values were used until such time as they could either be verified or modified.

The locations of sewage sampling and flow measurement recording stations within the Study Area were selected on the basis of providing the most representative and meaningful data for the design and evaluation of the proposed candidate systems for treatment. Every attempt was made to provide good areal distribution, to provide simultaneous

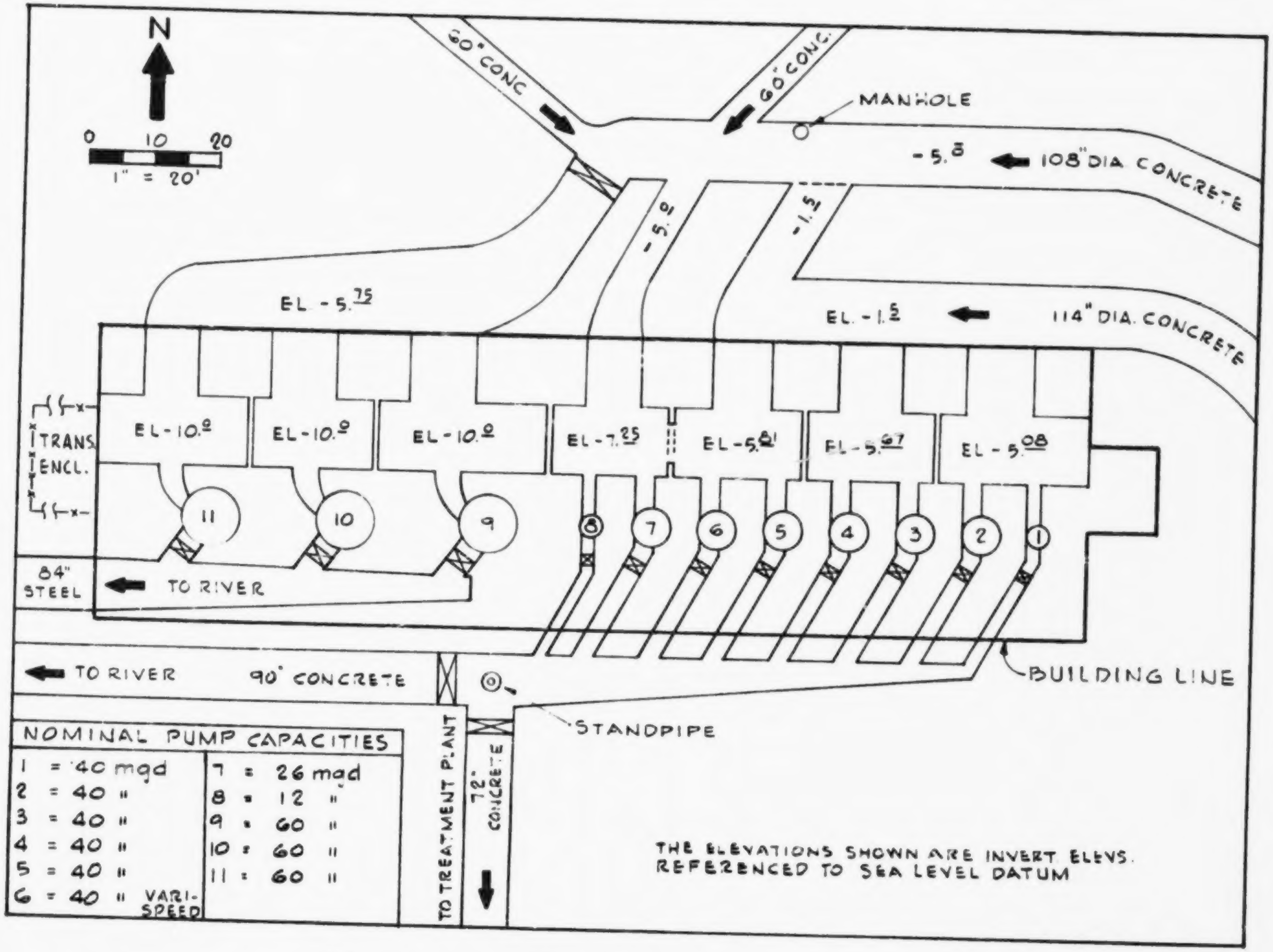
N
0 10 20
1" = 20'
60" CONC
60" CONC
MANHOLE
-5.8
108" DIA CONCRETE
-5.0
-1.5
EL - 5.75
EL - 1.5
114" DIA. CONCRETE
TRANS. ENCL.
EL-10.0
EL-10.0
EL-10.0
EL - 7.25
EL - 5.81
EL - 5.67
EL - 5.08
11
10
9
8
7
6
5
4
3
2
1
84" STEEL
TO RIVER
TO RIVER
90" CONCRETE
BUILDING LINE
STANDPIPE
TO TREATMENT PLANT
72" CONCRETE
NOMINAL PUMP CAPACITIES
1 = 40 mgd
2 = 40 "
3 = 40 "
4 = 40 "
5 = 40 "
6 = 40 " VARI-SPEED
7 = 26 mgd
8 = 12 "
9 = 60 "
10 = 60 "
11 = 60 "
THE ELEVATIONS SHOWN ARE INVERT ELEVS. REFERENCED TO SEA LEVEL DATUM

Figure 10. SCHEMATIC PLAN - SUMP NO. 2

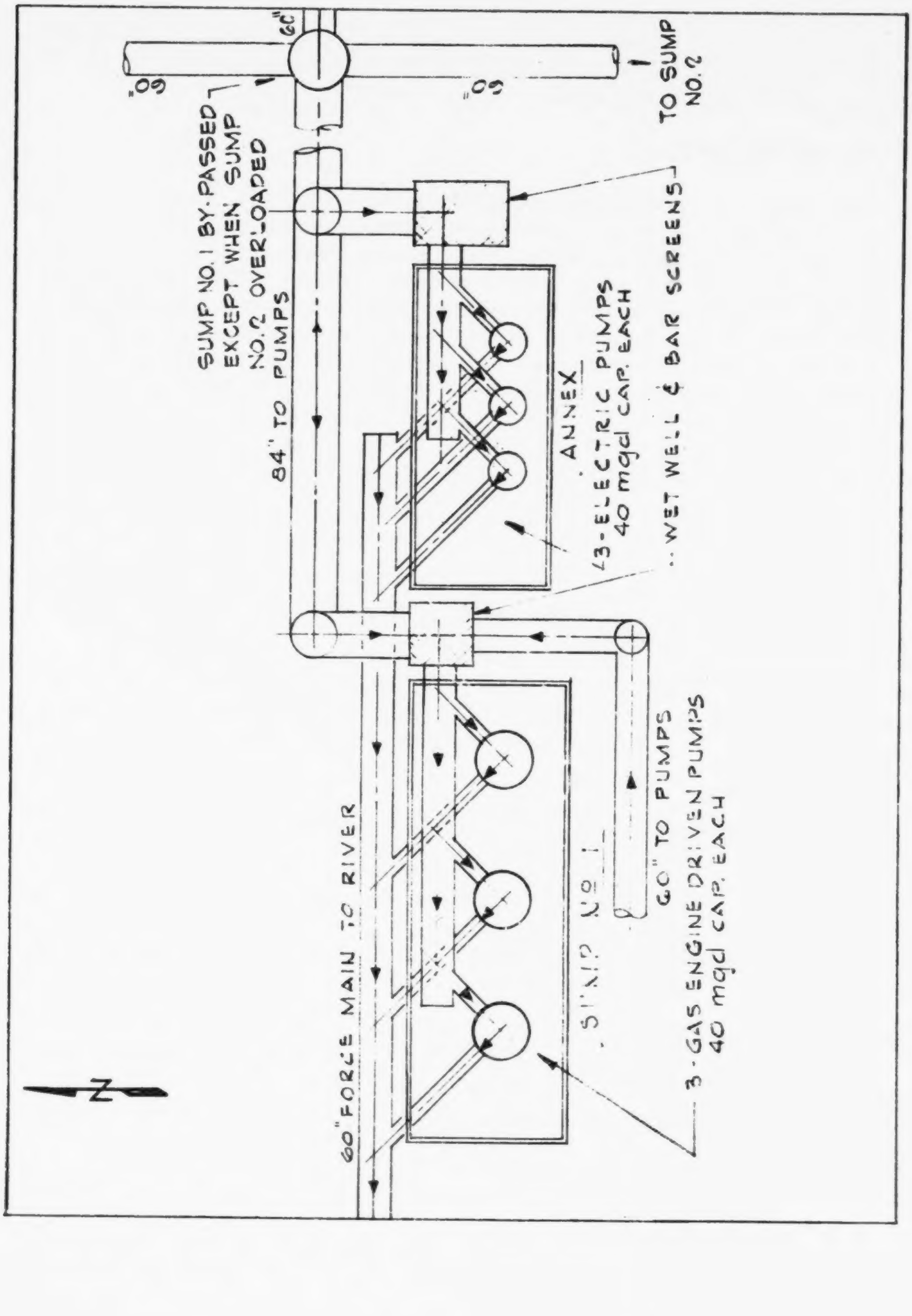
SUMP NO. 1 BY-PASSED EXCEPT WHEN SUMP NO. 2 OVERLOADED
60"
60"
60"
TO SUMP NO. 2
84" TO PUMPS
ANNEX
3-ELECTRIC PUMPS 40 mgd CAP. EACH
WET WELL & BAR SCREENS
60" TO PUMPS
SUMP NO. 1
3-GAS ENGINE DRIVEN PUMPS 40 mgd CAP. EACH
60" FORCE MAIN TO RIVER
N

Figure 11. SCHEMATIC PLAN - SUMP NO. 1

determination of quantities and compositions of the several discrete types of wastewater streams produced in or transported through the area, and to provide adequate information on the hydraulics of the present combined sewer system.

WASTEWATER SAMPLING

Sixteen sampling and measurement stations were selected to characterize combined sewage flow. Separated storm water runoff was observed at only one sampling station, which required three upstream measurement stations for proper flow determination. The sampling station for the separated storm water was situated at the junction of three storm sewers and provided a composite of the entering storm flows. Locations of wastewater sampling stations are shown in Figure 12. Stations 1 through 16 were situated on combined sewers and Stations 17, 17A, 17B, and 17C were located on separated storm sewers.

Station 1 was located approximately 600 feet upstream from Sump No. 2 on the 60-in. sewer line arriving from the vicinity of Sump No. 1. This sewer line is identified as Trunk D.

Station 2 was located on the 114-in. sewer line, identified as Trunk B, approximately 2,500 feet upstream from Sump No. 2. The sewage at this point is representative of combined flow from a predominantly residential area. Only single lines with relatively small drainage areas flow into the 114-in. pipe below the sampling station. All manholes on the 114-in. sewer between Sump No. 2 and Station 2 are permanently sealed because of the potential surcharge condition of the sewer.

Station 3 was located approximately 3,000 feet upstream from Sump No. 2 on the 108-in. sewer line identified as Trunk C. One 12-in. and one 18-in. line intercept the sewer between the sump and the sampling station. This sewer provides the major drainage from the Old City or gridiron-street-pattern portion of the Study Area. All manholes on the 108-in. sewer between Sump No. 2 and Station 3 are permanently sealed because of the potential surcharge condition of the sewer.

Station 4 was located on a 60-in. sewer line serving the southern and eastern sections of the Old City area, which together with another 60-in. pipe, is the principal contributor to Sump No. 1. Results from this station, in conjunction with Station 5, provided information on combined overflows at Sump No. 1.

Station 5 was located on a 60-in. sewer serving the central and western sections of the Old City area that comprises, with the addition of the sewer underlying Station 4, almost all the combined sewage flow from the Old City area that is not transported to Sump No. 2 via the 108-in. Trunk C sewer.

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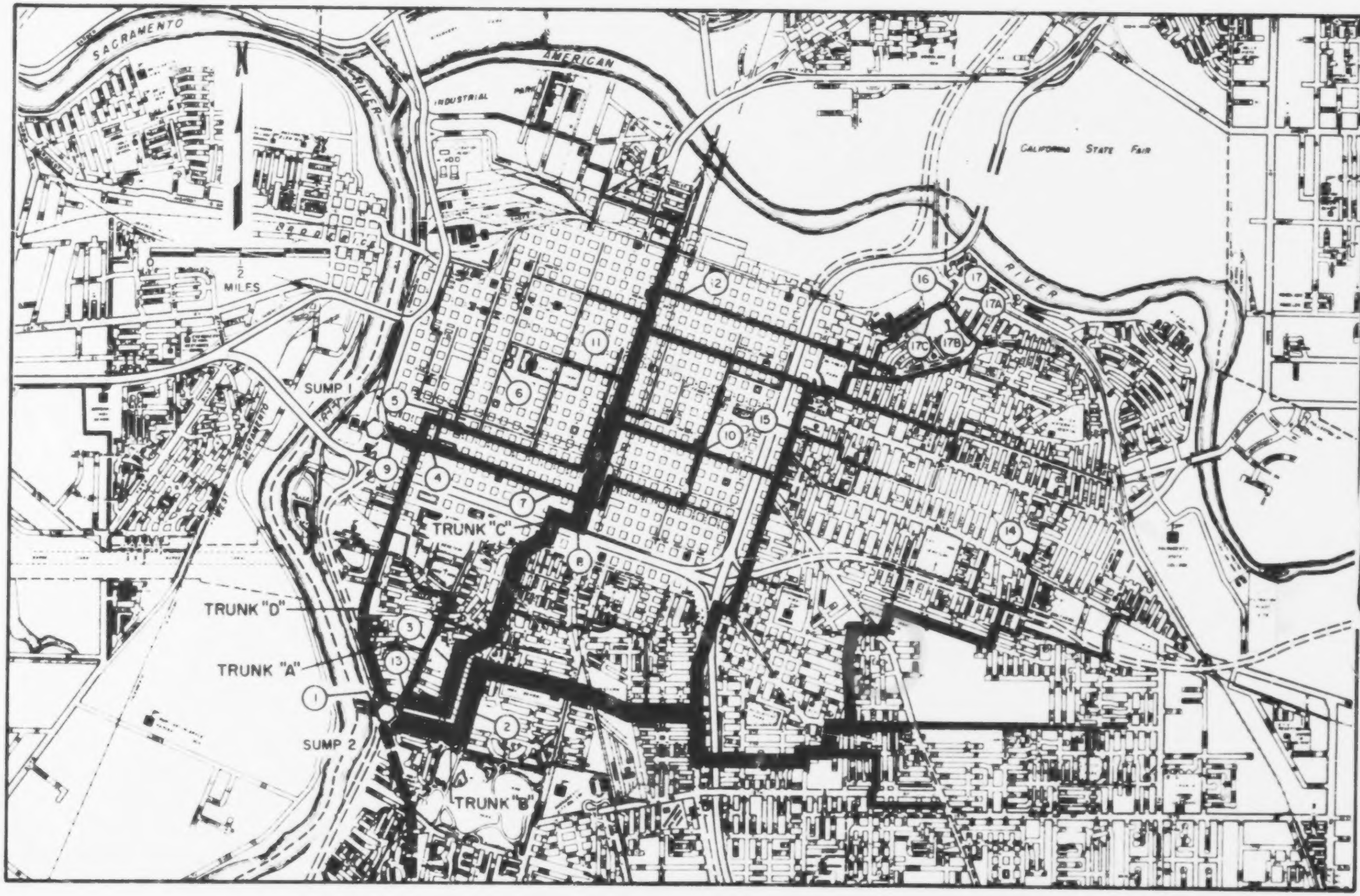
SACRAMENTO
RIVER
AMERICAN
RIVER
INDUSTRIAL
CALIFORNIA STATE FAIR
2 MILES
SUMP 1
SUMP 2
TRUNK "A"
TRUNK "B"
TRUNK "C"
TRUNK "D"
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
17A
17B
17C

Figure 12. LOCATION OF WASTEWATER SAMPLING STATIONS

Station 6 was chosen to establish realistic sanitary sewage flows and loading factors for the large transient or nonresidential employee population that exists in the Old City area. This station was located on a 24-in. sewer line that receives the 10-in. building connection directly at the sampling manhole. The building connection serves the State Resources Building, a large office facility.

Station 7 was placed on a 56-in. sewer line just downstream of a representative diversionary or relief structure wherein a smaller pipe, in this case the 56-in. line, intercepts at right angles and has a higher invert elevation than the larger 108-in. pipe with the entire smaller pipe section removed in the larger pipe. Thus, during low-flow conditions, all flow from the influent 56-in. line is diverted to the 108-in. sewer.

Station 8 was located on the 108-in. sewer line flowing from the Old City area immediately downstream of where it is intercepted by the 56-in. sewer monitored at Station 7. This station, coupled with Station 7, provided information on the composition and volume of the combined sewage.

Station 9 was located on the 84-in. sewer inlet to, and 150 feet upstream from, Sump No. 1. The monitoring of this station showed the flow conditions during dry-weather periods and the flow measurement and characteristics of combined sewage pumped by Sump No. 1 during overflow periods.

Station 10 was located near the terminus of a 36- by 42-in. brick elliptical combined sewer in the Old City area where it discharges into a 42-in. diameter sewer.

To characterize combined sewage flows generated in the commercial Old City area, Station 11 was located on a 33-in. sewer line at the northeast corner of Capitol Park upstream of its later connection with the large 108-in. collector.

Station 12 was located on a 45-in. sewer line which serves both a relatively small combined sewage area in the northeast section of Old City and a separate sanitary system in the northern part of the Study Area, in addition to the large exogenous sanitary sewage flow from County Sanitation District No. 2, situated across the American River to the North. This line has been observed on occasion to be surcharged.

Station 13 was located on a 60-in. inlet to Sump No. 2, approximately 1,000 feet upstream from the sump. This sewer line is identified as Trunk A. In conjunction with the other three trunk sewers tributary to Sump No. 2, (Trunks B, C, and D) the integrated characterization of the combined sewage produced in the total Study Area was obtained.

To characterize the separate sanitary sewage originating in a residential area within the Study Area, a sampling and measurement point was located on a 30-in. sewer at Station 14, upstream of any storm water contributions.

Station 15 was located on a 45-in. combined sewer that also services the sanitary flow from a residential area provided with a separate storm water system.

The sanitary sewage from the County Sanitation District No. 2, located north of the American River and outside the Study Area, was characterized at Station 16. Results obtained here on the composition of the sewage were used to also characterize the other significant exogenous sanitary flow from the North Sacramento-Hagginwood area, also outside the Study Area.

Station 17 was located on an 84-in. sewer line approximately 100 feet upstream from a lift station for the discharge of separated storm water runoff to the American River from a predominately residential area. This 84-in. storm sewer is fed by three separated storm sewers of 36-, 45-, and 48-in. diameter in close proximity to Station 17. Stations 17A, 17B, and 17C were located, respectively, on these tributary storm drains.

The characterization of the combined sewage and storm water during six wet-weather episodes was performed by collecting samples and measuring flows at each of the 19 sampling locations at, as nearly as practical, the commencement of rainfall, three hours thereafter, and approximately 12 to 18 hours after the commencement of sampling. Actual relations between the time of wet-weather wastewater sampling and the time and intensity of rainfall at the U. S. Weather Bureau rain gage situated at the U. S. Post Office in Sacramento are presented in Figure 13. To properly evaluate the variations in quantity and quality of sanitary sewage, the dry-weather measuring and sampling effort was conducted over a single continuous 24-hour period. Representative samples and measurements were taken at 2-hour intervals at 16 sampling stations. The 24-hour period, commencing on a Tuesday, was one that had been preceded by approximately 20 days with no recorded precipitation in the vicinity of the Study Area.

The wastewater flows were established at manhole sampling stations by measuring depth of flow at the station. Whenever this procedure was not possible due to surcharge conditions, the flows were determined from the hydraulic grade line, which was ascertained by measuring the depth of flow in the upstream and the downstream manholes. Relative rim invert elevations were established by field survey for all manholes, including those upstream and downstream, permitting the determination of flow depths from the distance between the water surface and the manhole rim. These data are presented in Table 2. The

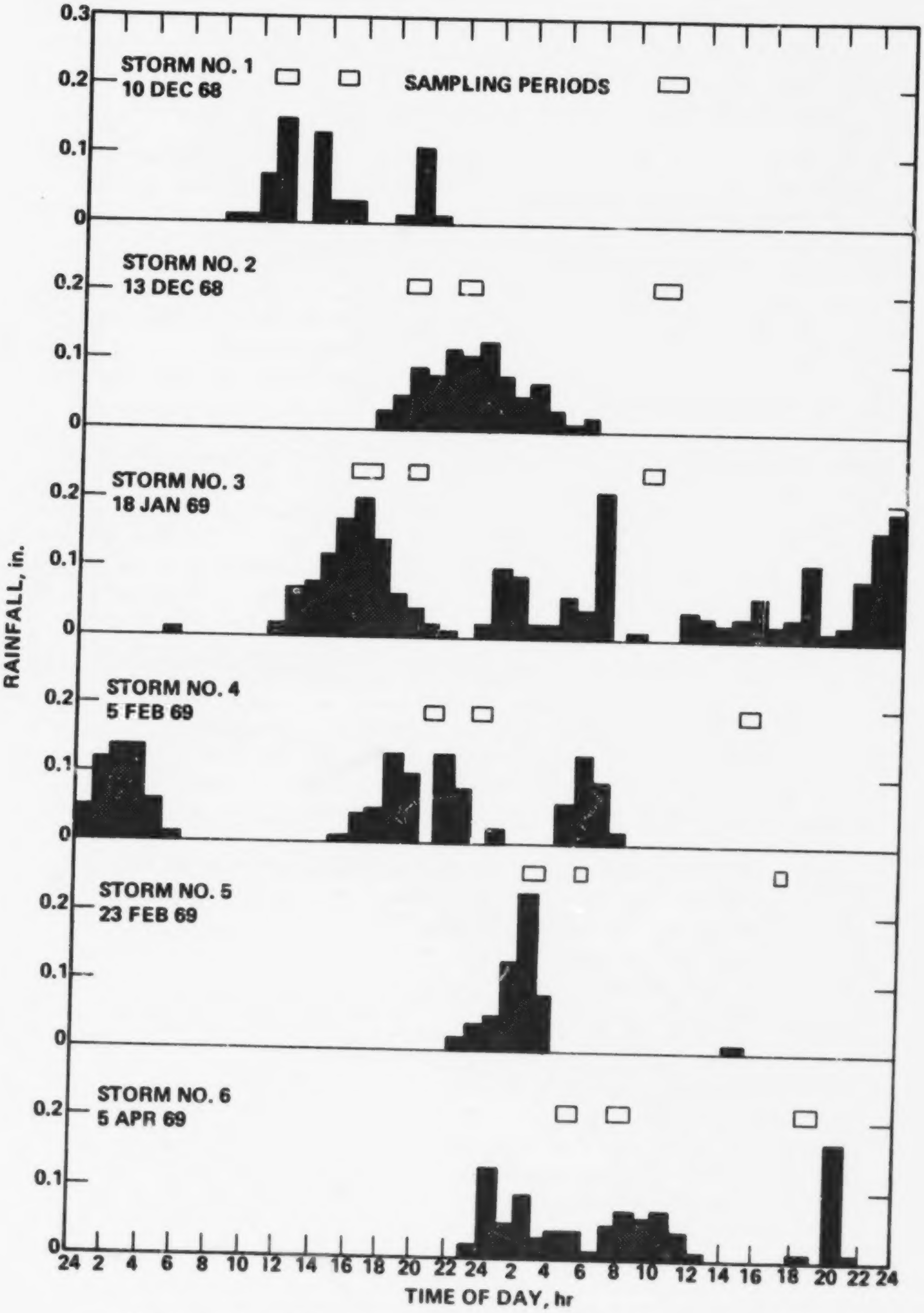

Figure 13. WET-WEATHER WASTEWATER SAMPLING PERIODS IN RELATION TO OBSERVED RAINFALL AT U.S. POST OFFICE

Table 2

SAMPLING STATION FIELD SURVEY DATA

Station	Rim Elev.[a]	Invert Elev.	Upstream Rim Elev.	Upstream Invert Elev.	Downstream Rim Elev.	Downstream Invert Elev.	Upstream Slope, %	Downstream Slope, %	Upstream Pipe Dia. in.	Downstream Pipe Dia. in.	Full Flow, cfs
1	57.18	35.18	63.47	35.89	-	-	0.064[b]	-	60	60	65.0
2	50.21	39.58	50.39	39.74	50.00	39.22	0.0426[b]	0.0815	114	114	300
3	51.12	36.34	51.72	36.65	50.13	36.05	0.0572[b]	0.0690	108	108	305
4	50.00	38.50	53.14	39.14	51.73	38.15	0.082[b]	0.092	60	60	73.0
5	50.00	36.60	50.82	36.72	51.73	38.15	0.063[b]	0.066	60	60	64.0
6	10-in. Pipe From State Water Resources Building										
7	47.59	35.84	50.00	36.00	48.24	35.49	0.042[b]	0.095	56	56	46.0
8	49.77	33.60	50.00	34.00	48.66	33.08	0.180[b]	0.180	108	108	520
9	50.00	35.10	50.01	36.01	51.03	34.61	0.252	0.204[b]	84	84	290
10	50.00	38.90	48.64	39.34	51.52	38.72	0.105	0.080[b]	42	42	28.0
11	50.00	39.30	49.79	39.79	50.80	39.10	0.130	0.051[b]	30	33	12.0
12	50.00	34.75	48.78	35.36	49.93	33.34	0.054[b]	0.176	45	45	28.0
13	55.38	36.88	54.98	36.91	55.75	36.50	0.272	0.112[b]	60	60	85.0
14	50.00	43.40	50.40	43.70	50.49	43.29	0.130	0.046	24	24	4.8
15	50.00	36.09	45.99	36.24	47.27	35.97	0.043	0.034[b]	45	45	22.0
16	50.00	40.70	-	-	49.76	39.76	-	0.52[b]	24	24	16.0
17A	50.00	37.30	50.29	37.49	48.94	24.24	0.087[b]	1.03	36	36	19.0
17B	48.17	29.17	47.39	29.72	51.03	28.03	0.172[b]	0.247	45	45	50.0
17C[c]	48.39	32.39	46.37	32.37	51.03	28.03	-0.0067	1.62	48	48	

[a] All elevations are relative

[b] Slopes used to calculate full pipe flows

[c] Hydraulic gradient slope to be used to calculate flow at station 17C

flow was calculated from these measurements using Manning's equation, $q = 1.49ar^{0.67}S^{0.5}/n$, where q is flow in cfs, a is wetted cross-sectional area in square feet, r is wetted hydraulic radius in feet, S is slope of water surface or invert, and n is roughness coefficient. The roughness coefficient was assumed to be equal to 0.013, a design value used by the City of Sacramento Engineering Department.

All samples collected during both dry-weather and wet-weather episodes were analyzed for total and volatile suspended solids, settleable solids, biochemical oxygen demand, chemical oxygen demand, and fecal coliform concentrations and pH value.

PRECIPITATION

Rainfall in the Study Area was characterized by the U. S. Weather Bureau at the most representative reporting weather station. The location of the reporting station changed several times during the 18 years of continuous rainfall record from June 1950 to June 1968 that was applied to the unit hydrograph. For the period from 1 June 1950 through 19 November 1958, it was located at the Post Office at 9th and I Streets in downtown Sacramento. Thereafter it was located 1.5 miles to the southeast at 23rd and R Streets until 29 September 1964, at which time it was returned to the Post Office at 9th and I Streets. Beginning with 1 January 1964 and throughout the remainder of the continuous record used in this program, the municipal airport, located 4.5 miles to the south of the Post Office, was the reporting station. The several rain gage locations and their relation to the Study Area are shown in Figure 14.

Total daily rainfall recorded at the Post Office during the period of the sewage characterization program is presented in Figure 15.

MUNICIPAL SEWAGE

The overall flow and composition of municipal sewage converging at Sump No. 2 could not be ascertained directly. Sump No. 2 is used to control flow discharge into the Main Treatment Plant, resulting in rather frequent surcharging of the lower tributary trunk sewers. It was found that this nonuniform flow condition due to storage occurred for about 1.5 to 2 miles upstream from Sump No. 2.

Mean daily flows are quite variable even during dry-weather episodes due to, among other things, extensive and excessive use of water for lawn and shrubbery irrigation. The latest available records (Ref. 4) on the average monthly discharge rates from Sump No. 2 to the Main Treatment Plant indicate that for the summer months of June 1967 and July, August, and September 1966 the sewage flows were 43.5, 40.3, 57.7, and 54.5 mgd, respectively. Corresponding monthly rainfalls

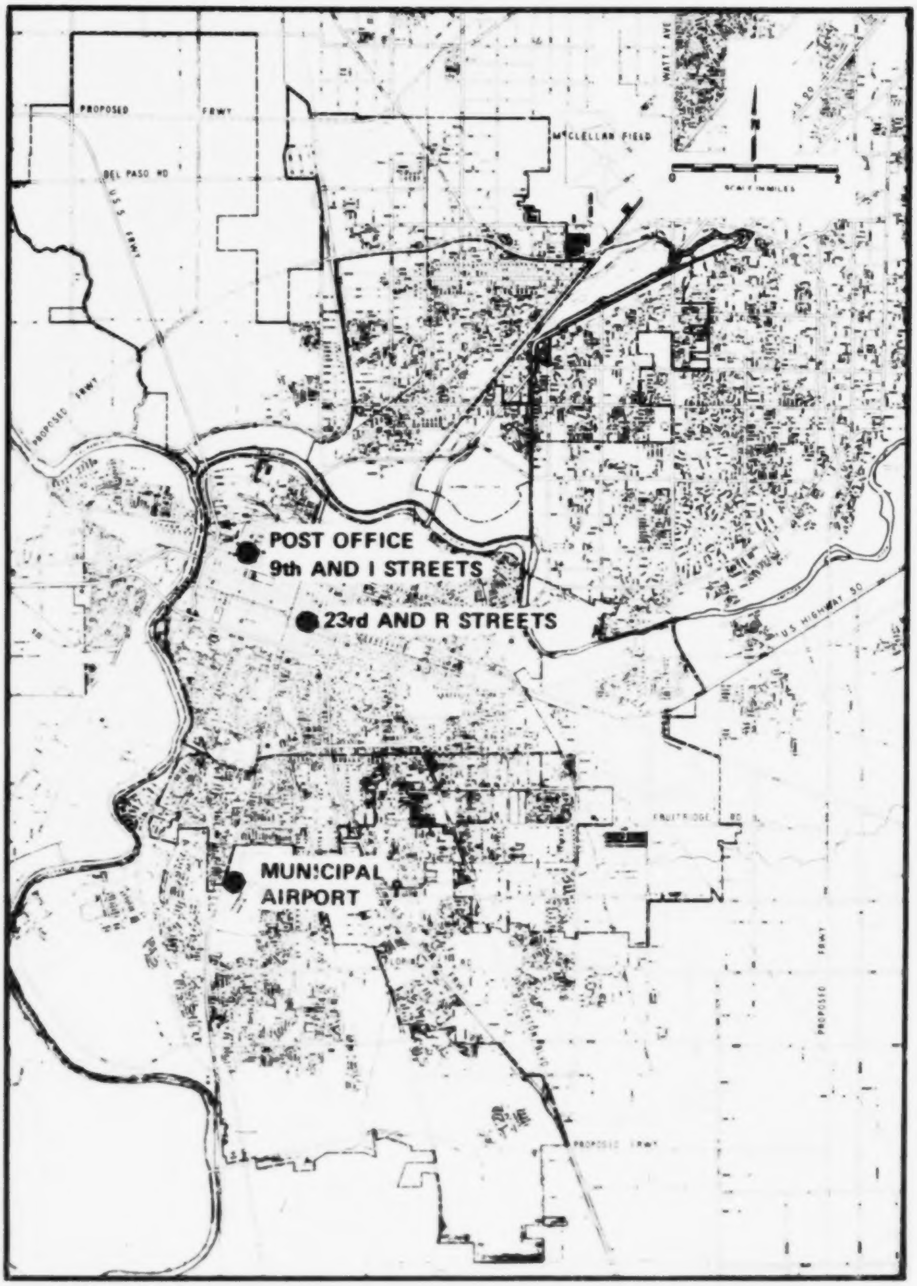
PROPOSED FRWY
DEL PASO RD
McCLELLAN FIELD
WATT AVE
SCALE IN MILES
0
2
PROPOSED FRWY
POST OFFICE
9th AND I STREETS
23rd AND R STREETS
US HIGHWAY 50
FRUITRIDGE RD
MUNICIPAL AIRPORT
PROPOSED FRWY
PROPOSED FRWY

Figure 14. LOCATION OF RAIN GAGES

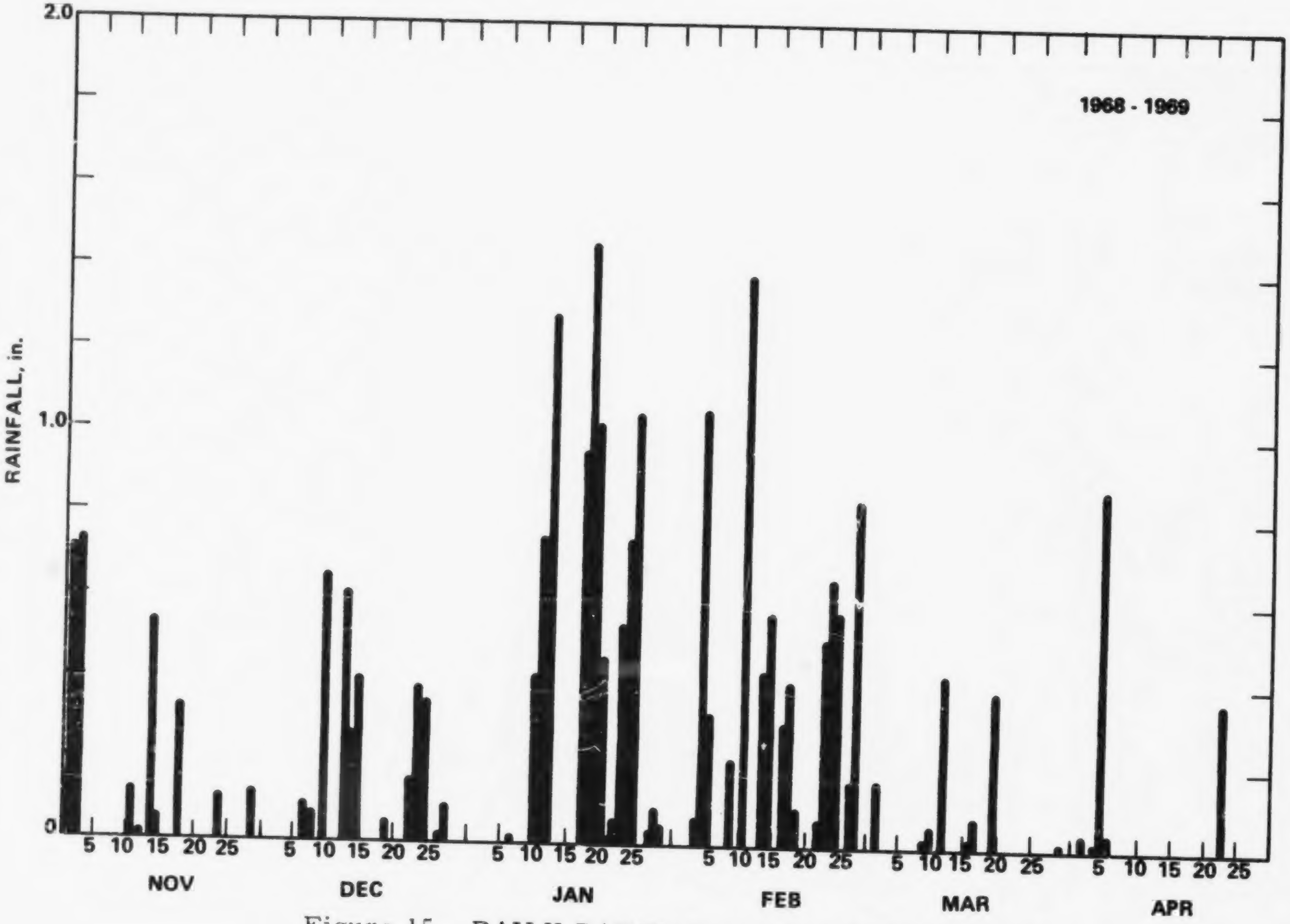

Figure 15. DAILY RAINFALL - U.S. POST OFFICE

as recorded at the Municipal Airport were 0. 60, 0. 10, 0. 00, and 0. 07 inches, respectively. Diurnal variations in the dry-weather sanitary sewage characteristics were established on the basis of analysis of stations upstream from the influence of Sump No. 2. Figure 16 shows the observed diurnal variation in dry-weather flow and composition at Sampling Station 8, which is considered representative of other sampling stations. The major peak occurs approximately in the middle of the afternoon and a smaller peak occurs subsequently in the middle of the evening. Minimum flow occurs during early morning hours. The average daily flow at this sampling station was approximately 45 mgd; the peak flow was about 140 percent of average and minimum flow about 50 percent of average.

A similar diurnal variation of dry-weather sewage flow and quality is shown in Figure 17. The peak flow from this large densely populated commercial structure occurs at 2:00 p. m. , with the minimum flow (no flow) occurring at approximately 3:00 a. m. Figure 17 shows that the greatest volume of flow from this building occurs, as expected, during normal working hours.

The principal industrial wastes generated in Sacramento from the standpoint of wastewater volume and strength are from food processing operations. A City ordinance requires preliminary screening prior to discharge into trunk sewers, but these process wastes comprise a major portion of the load at the Main Treatment Plant. The food process wastes are basically seasonal and occur primarily during late summer and autumn. Figure 18 shows the location of the major industrial wastewater discharges in the Study Area.

Complete dry-weather results characterizing municipal sewage are presented in Appendix A.

SEPARATED STORM WATER RUNOFF

Water quality characteristics of the storm water runoff collected from a separated storm sewer system are presented in Table 3. Reliable data on corresponding flows could not be ascertained due to difficulties encountered in measuring the depths at the selected location (Stations 17, 17A, 17B, and 17C).

The magnitudes of the various wastewater constituents were found to be variable with respect to both date and time, but certain trends were indicated.

The concentrations diminished as the wet season progressed and became of more equal value during the course of a single rainfall event. Also the somewhat higher concentrations of BOD and COD observed during the last sampling episode in April were no doubt a consequence of the relatively dry preceding month (see Figure 15).

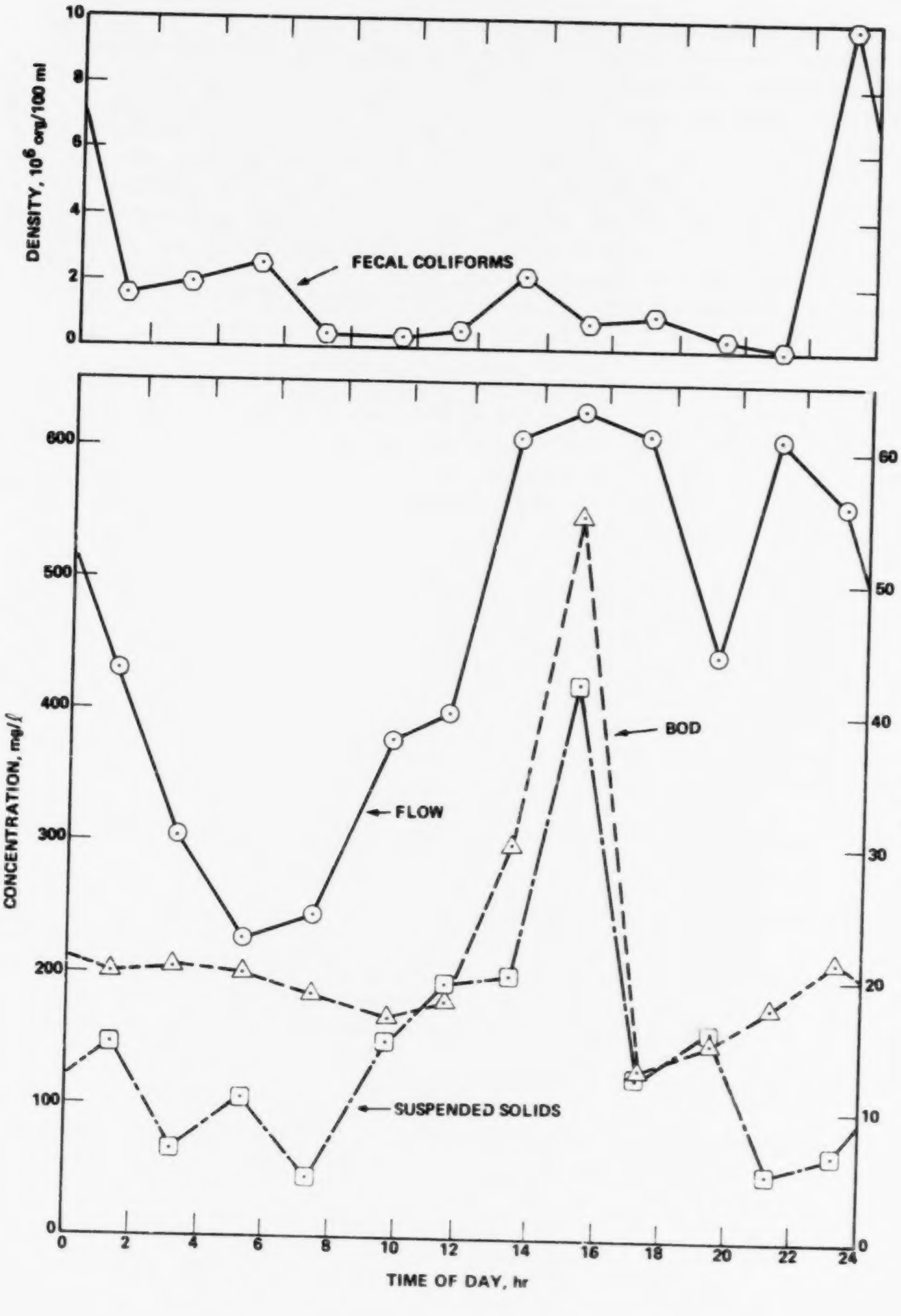

Figure 16. DIURNAL MUNICIPAL SEWAGE CHARACTERISTICS AT STATION 8

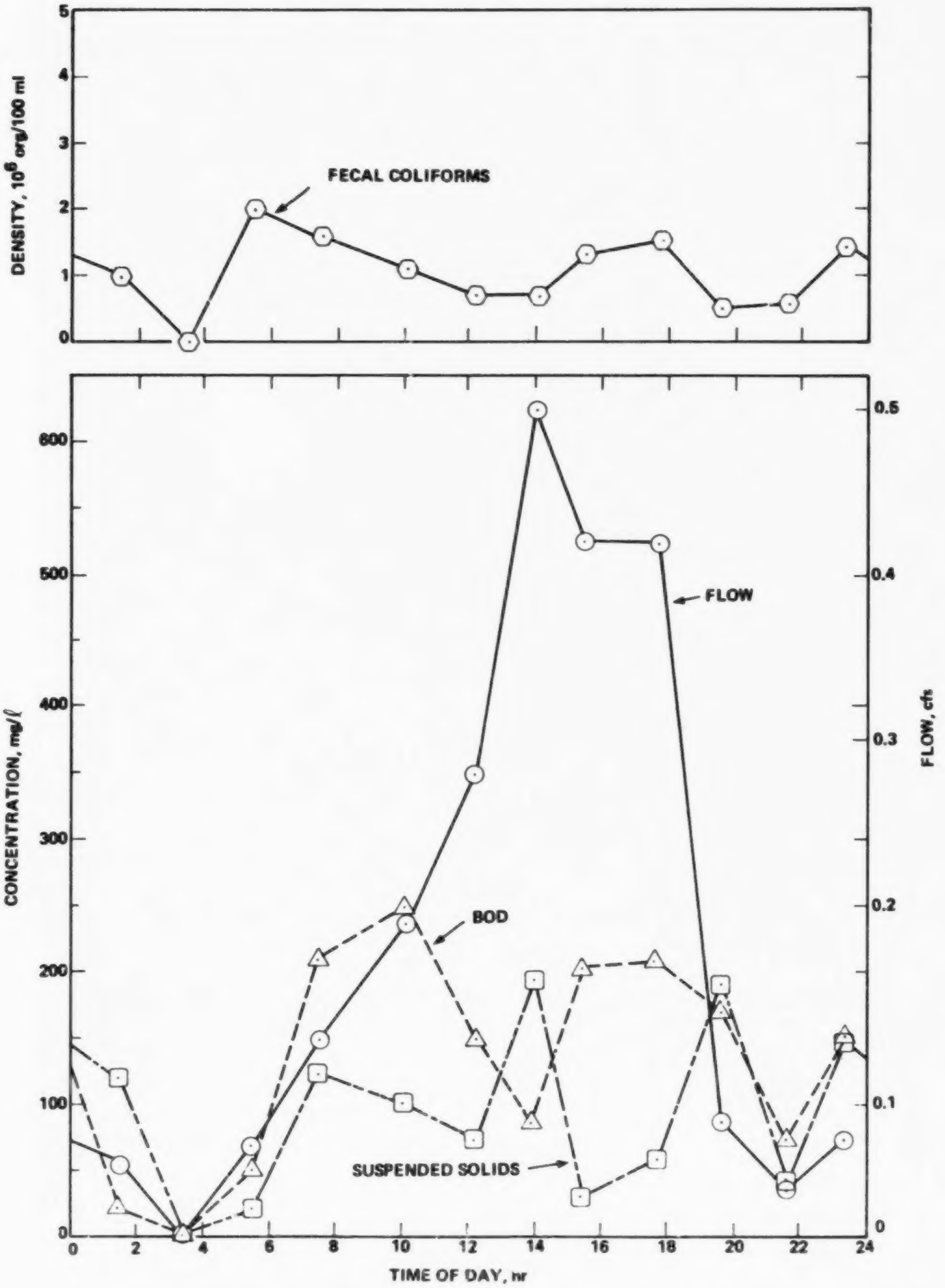

Figure 17. DIURNAL CHARACTERISTICS OF SEWAGE FROM STATE WATER RESOURCES BUILDING

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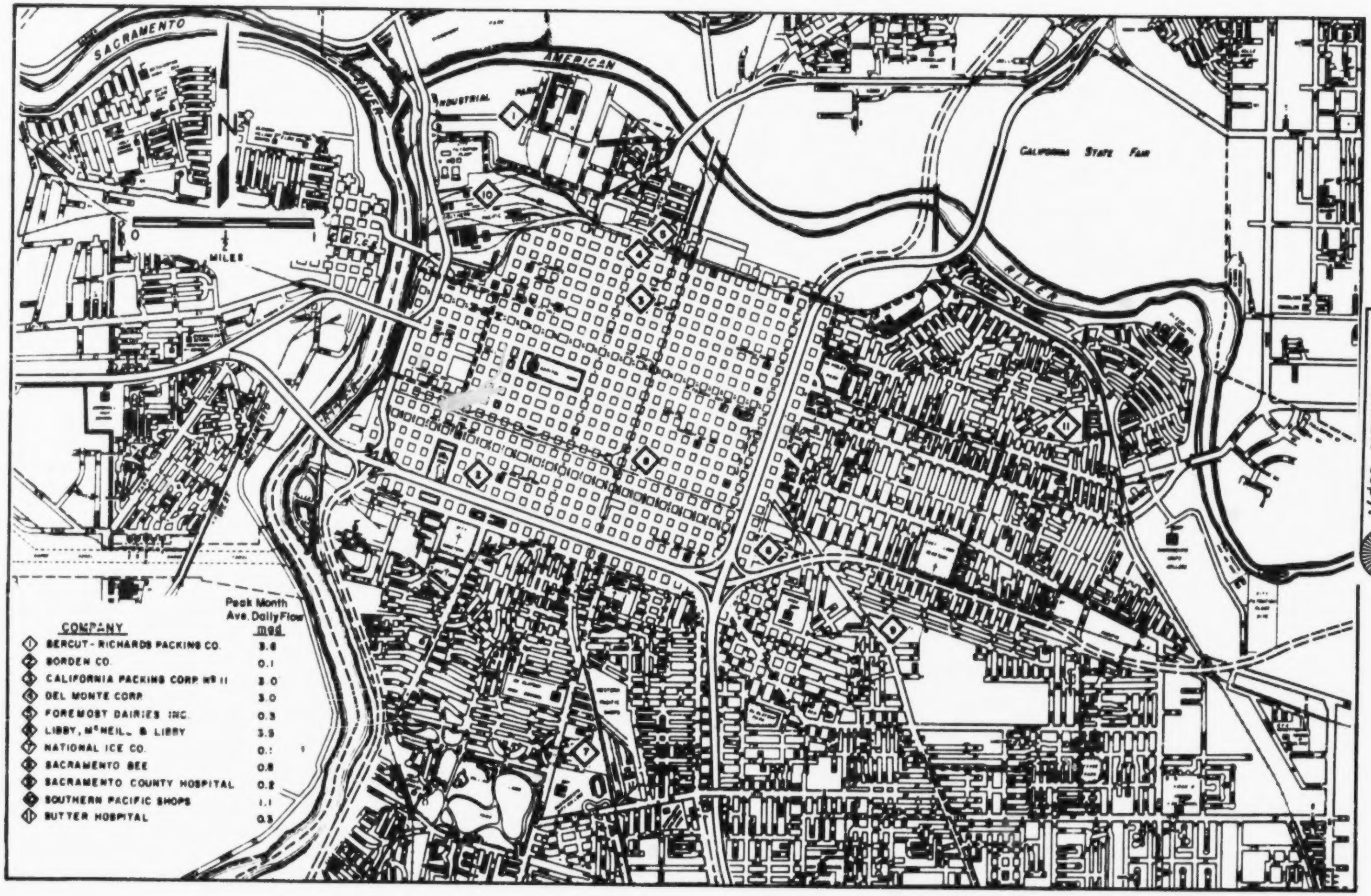
SACRAMENTO
RIVER
AMERICAN
RIVER
INDUSTRIAL
CALIFORNIA STATE FAIR
N
0
1
MILES
1
2
3
4
5
6
7
8
9
10
11
COMPANY
Peak Month Ave. Daily Flow mgd
1 BERCUT-RICHARDS PACKING CO. 3.8
2 BORDEN CO. 0.1
3 CALIFORNIA PACKING CORP. Nº 11 3.0
4 DEL MONTE CORP. 3.0
5 FOREMOST DAIRIES INC. 0.3
6 LIBBY, McNEILL & LIBBY 3.5
7 NATIONAL ICE CO. 0.1
8 SACRAMENTO BEE 0.8
9 SACRAMENTO COUNTY HOSPITAL 0.2
10 SOUTHERN PACIFIC SHOPS 1.1
11 SUTTER HOSPITAL 0.3

Figure 18. LOCATION OF MAJOR INDUSTRIAL WASTEWATER DISCHARGES

Table 3

SEPARATED STORM WATER RUNOFF QUALITY CHARACTERISTICS - STATION 17

Characteristic	Sampling Sequence*	Date					
		10 Dec 68	13 Dec 68	18 Jan 69	5 Feb 69	23 Feb 69	5 Apr 69
Total Suspended Solids, mg/ℓ	1	54	48	171	26	27	25
	2	208	211	43	38	51	19
	3	22	3	134	35	46	45
Volatile Suspended Solids, mg/ℓ	1	54	30	39	10	21	25
	2	208	211	23	19	3	19
	3	18	3	37	17	22	35
Settleable Solids, mg/ℓ	1	52	48	96	25	25	14
	2	193	187	31	18	35	15
	3	22	0	115	19	38	43
BOD, mg/ℓ	1	52	92	260	52	24	283
	2	118	117	194	46	39	274
	3	67	105	65	86	39	131
COD, mg/ℓ	1	110	66	40	45	44	77
	2	176	46	38	66	27	136
	3	36	21	44	28	45	36
pH	1	6.6	6.8	7.2	7.3	7.2	6.8
	2	6.6	7.6	6.9	7.3	7.3	6.9
	3	7.2	7.0	7.0	7.5	7.6	7.3
Fecal Coliforms, org/ℓ	1	2.9×10^5	2.0×10^6	5.5×10^4	8.0×10^4	6.6×10^4	6.0×10^6
	2	7.5×10^5	2.0×10^6	1.3×10^5	8.0×10^4	4.0×10^5	2.0×10^6
	3	1.0×10^7	4.0×10^6	2.4×10^4	8.0×10^4	4.0×10^5	4.0×10^5

*1 denotes as nearly as practical to start of storm, 2 denotes three hours thereafter, and 3 denotes 12 to 18 hours after the commencement of sampling.

COMBINED SEWAGE

Results of the wet-weather measurement, sampling, and analysis at Station 8, which is typical of the overall combined sewage from the Study Area, are presented in Table 4. It is evident that the concentrations of wastewater constituents are highly variable and change quite rapidly as a result of different admixtures of storm water runoff and sanitary sewage, whose compositions and quantities are extremely time dependent.

Complete data on the wet-weather monitoring of the combined sewage system are given in Appendix B.

RECEIVING WATER CHARACTERISTICS

The eventual recipient of all wastewaters from the Study Area is the Sacramento River, which flows past Sacramento to the San Francisco Bay system and thence to the Pacific Ocean through the Sacramento-San Joaquin Delta. The drainage basin upstream of the City of Sacramento has an area of approximately 24,000 square miles and contributes about 65 percent of the flows to the Delta (Ref. 5). Major tributaries are the Feather, Yuba, and American Rivers. The American River bounds the Study Area on the north before joining the Sacramento River, which bounds the Area's western extremity.

Many factors affect the flow and quality of the Sacramento River at Sacramento, including storm water runoff, reservoir water releases and storage, water diversions for use and exportation, power generation, and lunar tides. Even though the stream discharges are becoming more extensively regulated by reservoir systems, the flows past Sacramento are still expected to have great variation in the future, ranging from perhaps 5,000 cfs in the spring to in the order of 50,000 cfs during the winter rainy season.

Water quality objectives have been adopted to protect and preserve the many beneficial uses of the water of the Sacramento-San Joaquin Delta, which includes at its northern extremity the Sacramento River at the City of Sacramento. These beneficial water uses include domestic and municipal, agricultural, and industrial supply; propagation, sustenance, and harvest of fish, aquatic life, and wildlife; recreation; aesthetic enjoyment; and navigation. In addition, the discharge of wastewaters into these receiving waters for waste assimilation, transport, and diminution was considered as a beneficial use in the formulation of Delta water quality objectives.

Many factors influence or describe receiving water quality, such as color, odor, floating grease and oils, benthic deposits, bacteria, trace elements, temperature, pH, radioactivity, turbidity, dissolved oxygen, nitrogen forms, biocides, total dissolved solids, chlorides,

Table 4

COMBINED SEWAGE CHARACTERISTICS - STATION 8

Characteristic	Sampling Sequence*	Date					
		10 Dec 68	13 Dec 68	18 Jan 69	5 Feb 69	23 Feb 69	5 Apr 69
Flow, cfs	1	218	117	-	237	242	88
	2	188	291	233	223	150	61
	3	52	62	281	72	74	40
Total Suspended Solids, mg/ℓ	1	227	216	502	140	56	242
	2	47	53	186	110	151	91
	3	230	30	66	98	208	241
Volatile Suspended Solids, mg/ℓ	1	164	173	311	56	51	242
	2	26	53	186	82	112	91
	3	60	30	26	98	162	221
Settleable Solids, mg/ℓ	1	28	159	488	127	34	233
	2	0	53	111	106	142	52
	3	119	28	44	70	202	210
BOD, mg/ℓ	1	74	193	241	808	75	306
	2	197	160	258	132	73	70
	3	171	328	175	186	69	-
COD, mg/ℓ	1	387	380	513	160	195	191
	2	234	146	257	176	191	123
	3	291	431	59	285	318	354
pH	1	6.9	7.0	7.1	7.2	7.5	7.0
	2	7.1	6.9	6.8	6.9	7.5	6.6
	3	7.3	7.2	7.0	7.2	7.1	6.5
Fecal Coliforms, org/ℓ	1	2.4×10^6	3.0×10^7	5.9×10^6	2.0×10^6	1.2×10^6	2.0×10^6
	2	2.6×10^6	8.6×10^7	1.6×10^6	1.6×10^6	6.6×10^5	4.5×10^6
	3	6.6×10^7	1.2×10^7	8.9×10^5	7.0×10^6	7.0×10^6	1.2×10^7

*1 denotes as nearly as practical to start of storm, 2 denotes three hours thereafter, and 3 denotes 12 to 18 hours after the commencement of sampling.

and toxic materials. Of the wastewater quality characteristics that constitute or affect these factors, total suspended solids, BOD, and fecal coliforms were chosen as being adequate for describing and evaluating a system for the control of storm water runoff and combined sewage overflows, at least within the context and scope of this program.

The concentrations of these constituents indigenous to the Sacramento River at Sacramento are dependent on, among other things, stream flow. Figure 19 presents suspended sediment (solids) concentrations reported (Ref. 6) for the Sacramento River at Sacramento for the three water years extending from October 1963 to September 1966, from which a correlation between suspended solids and stream discharge has been drawn. Suspended solids concentrations reported for the 1968-69 water year, which were not available for use during the normal conduct of this program, provide generally lower suspended sediment loads at flows in excess of 25,000 cfs than are represented by Figure 19, and may be indicative of the ever-increasing upstream flow regulation of the Rivers. Calculations indicate however that the use of these more recent data would have negligible effects on the findings of this study.

BOD concentrations representative of background levels were not available over most of the range of river flows experienced, but the limited data that were obtainable indicated values ranging from 1 to 7 mg/l of BOD for flows between 7,000 and 10,000 cfs, with most values being 1 to 2 mg/l. In an investigation (Ref. 7) by Hydroscience, Inc. for the County Department of Public Works on the pollution assimilative capacity of the lower Sacramento River, it was stated that the background values of BOD are likely to increase to 2.0 mg/l.

Historical fecal coliform densities were not available for the Sacramento River, but samples taken from the I-Street Bridge during the conduct of this program indicated upon analysis an inverse relation between fecal coliform densities and River flows ranging from 10,000 to 75,000 cfs as shown in Figure 20.

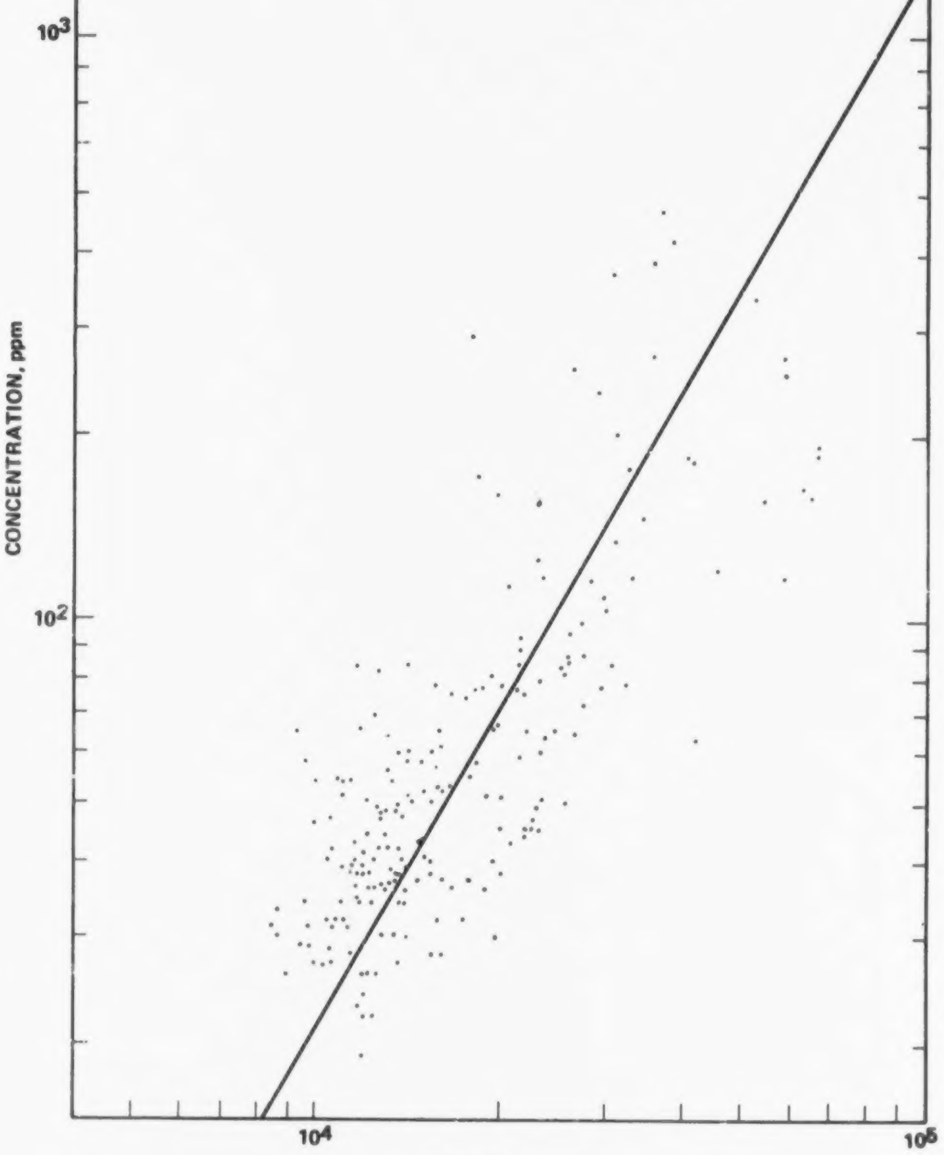

Figure 19. RELATIONSHIP BETWEEN RIVER FLOW AND SUSPENDED SOLIDS CONCENTRATION AT SACRAMENTO

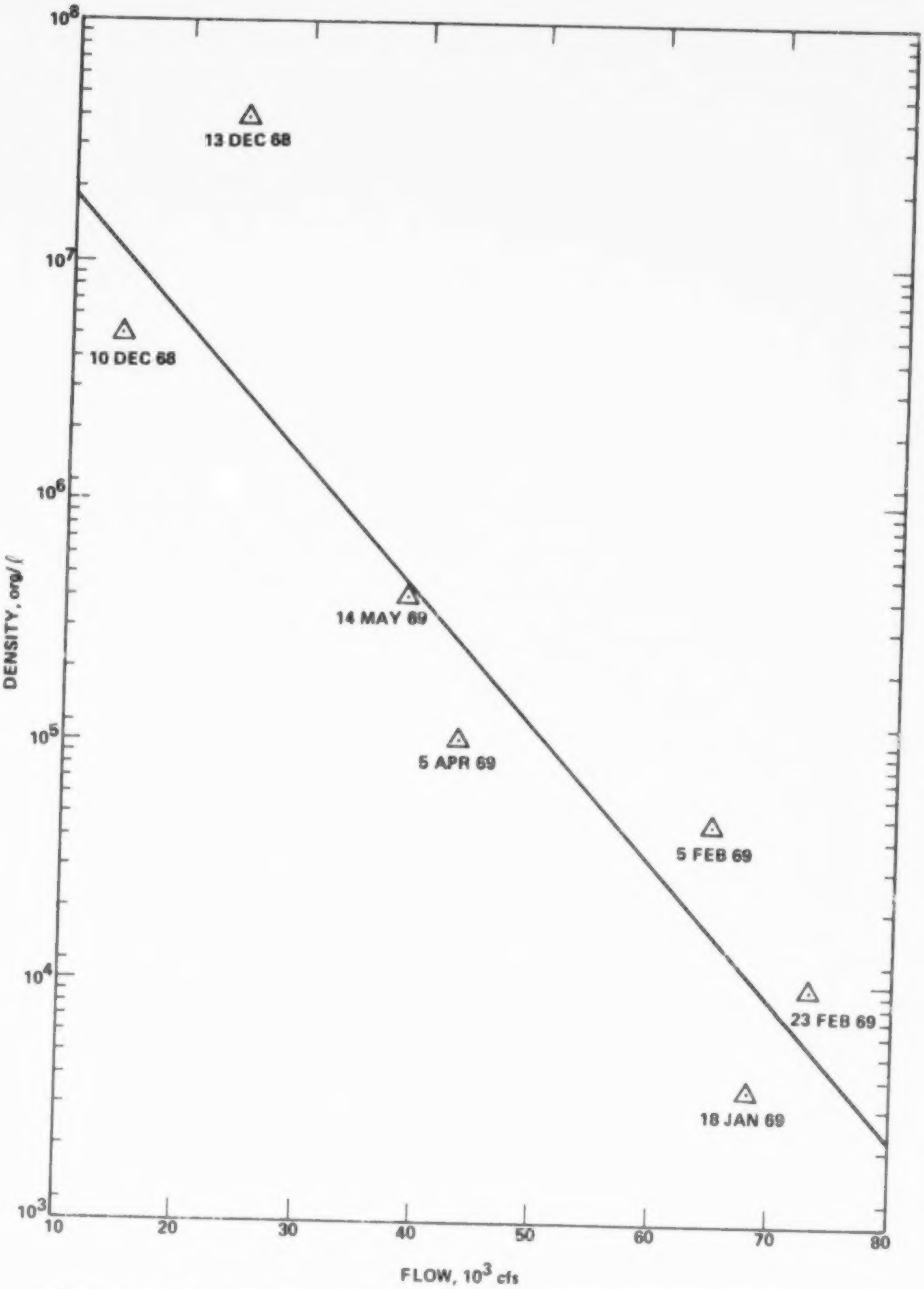

Figure 20. RELATIONSHIP BETWEEN RIVER FLOW AND FECAL COLIFORMS CONCENTRATION AT SACRAMENTO

Section VI

SYSTEMS DESIGN

Development and design of implementable systems for the control of storm water runoff and combined sewer overflows is complicated by many diverse and sometimes contradictory factors. Upon determination of controlling physical conditions relevant to the problem, such as wastewater quantities and qualities, receiving water quality and assimilative capacity, and existing sewerage facilities and appurtenances, candidate or alternative systems must be conceived and developed in accordance with specific design parameters and criteria. In addition, the systems must be formulated within the constraints of realistic phasing for the implementation of the systems over a future time period, of applying reasonable financing methods to this schedule, and of providing in some cases a suitable organization or agency to effectively administer the construction, operation, and maintenance of the systems.

SYSTEMS DEVELOPMENT

Storm water runoff and combined sewer overflow pollution control systems were based on pre-selected concepts that demonstrated promise of meeting expected requirements both because of proven performance or recent technological advances and on concepts that represented a wide range of solutions applicable not only to the Sacramento area but throughout the nation.

CANDIDATE SYSTEMS CLASSIFICATION

Seven basic candidate systems were investigated in this study. These systems were inter-related in varying degrees since it became necessary to combine parts of several of the candidate systems to achieve a complete sewer system consisting of wastewater collection, conveyance, treatment, and disposal.

The seven basic candidate systems or concepts are identified by name as Complete Separation, ASCE Force Main, Surface Storage, Underground Storage, Stabilization Pond, Dissolved Air Flotation, and Mechanical Screening.

Complete Separation considered total separation of sanitary and storm flows. This entailed the installation of a new gravity conveyance system for either sanitary or storm flows with the existing system then utilized for the other.

ASCE Force Main also necessitated total separation of sanitary and storm flows. The basic concept called for the sanitary flow to be managed through force mains located within the existing sewers, which would continue to convey the then separated storm flows. This system was divided further into two parts in order to explore an alternative which appeared well suited for application in Sacramento. Total Force Main was in full accord with the ASCE concept wherein each home was served by an individual grinder-pump unit. The other system, Hybrid Gravity-Force Main, considered the use of larger zonal grinder-pump stations with the sanitary sewage delivered to the station via conventional gravity lines.

Surface Storage and Underground Storage provided means for retaining peak flows and thereby modulating the burden on pumping and treatment facilities. The latter system envisioned near-surface storage.

Stabilization Pond also entailed storage, but was considered primarily as a treatment process in itself rather than as a reservoir for retention of storm water runoff or combined sewage flows prior to treatment. Dissolved Air Flotation and Mechanical Screening were two other treatment processes evaluated.

A complete wastewater system can be comprised of many possible combinations of candidate systems and other components. The system design is further complicated due to spatial and temporal constraints imposed by a particular study area.

To assist in the systems development, the work was arranged into four main groupings within which similar characteristics existed. These four major subsystems were Collection and Conveyance, Storage, Treatment, and Disposal. Each of the candidate systems under investigation fell under one of the first three classifications above. While the actual employment of a storage subsystem or a treatment subsystem was considered as optional, the development of any complete wastewater system would require consideration of all four major subsystems.

PLANNING HORIZON

All systems investigated in this program were predicated on meeting projected requirements for 20 years after the start of construction. By establishing 1972 as the earliest practical year that a selected system could realistically be started, the 20-year life would extend the planning horizon to 1992. Then, when system capacity was theoretically reached, it was assumed that additional capacity could be provided either by parallel systems or, more likely, by new methods developed through technological advances that had occurred during the intervening years.

As a practical matter it was recognized that any selected improvement program of the magnitude envisioned in this study would be construction phased over several years. The intent here has been to provide a reasonable basis for defining and costing the alternative systems on an equal basis.

DESIGN CRITERIA

Design criteria were established on the basis of present practice and projections of current data to the year 1992. Present data were acquired both through available sources and the conduct of field measurement programs in the Study Area.

WASTEWATER CHARACTERISTICS

Only sanitary sewage flows that would occur in a completely separated sanitary and storm sewer system were considered in this program. Whereas present dry-weather sewage flow rates are quite seasonal due to the collection of extensive lawn irrigation return water and cooling system discharges in the existing combined sewers, these wastewaters would not be expected to enter a sewer used exclusively for sanitary sewage. During wet-weather episodes these extraneous wastewaters would not be prevalent and the respective quantities of combined sewage would consist solely of sanitary sewage and storm water runoff.

Sanitary Sewage

Sanitary sewage flows for the Study Area were derived using projected residential and employee populations, anticipated population distribution, and per capita wastewater discharge factors. The only exceptions were several major industrial discharges and the exogenous sanitary sewage from County Sanitation District No. 2, which is presently fixed in magnitude by contract and expected to remain so.

The residential population for the Study Area and the North Sacramento-Hagginwood area was projected to the year 1992 utilizing basic data furnished by the Sacramento City Planning Department. The County Planning Department and the Sacramento Chamber of Commerce supplied basic data for the projected employee population within the Study Area and the North Sacramento-Hagginwood area, excluding State employees and the student population at Sacramento State College. The latter two were projected populations from data furnished by the State and the College, respectively.

Projected 1992 populations for the Study Area and North Sacramento-Hagginwood area were 201,500 residents and 176,200 employees, including State personnel and student population at Sacramento State College, providing a total population of 377,700 for both the Study Area and the North Sacramento-Hagginwood area.

The projected residential population was distributed over the Study Area in relation to the future anticipated zoning. This distribution was accomplished by assigning a 1992 residential population to each 1968 census tract and its respective blocks based on an area ratio for each tract. All future anticipated nonresidential areas were excluded in the distribution.

The projected employee population was distributed over the Study Area in accordance with future anticipated commercial and industrial zoning. Here again, the census tracts and the respective blocks were utilized for distribution of the 1992 employee population with a greater percentage of employee population assigned to the larger blocks in each tract. State employees were assigned on the basis of individual building floor area as proposed in the California State Capital Plan. The proposed building locations have been tentatively fixed so that the individual block or blocks in each tract were known.

The average per capita sewage flow established for the Study Area was 85 gpd for residents and 40 gpd for employees. The residential waste discharge factor of 85 gpd/cap was based on the ultimate average design flow established in a study (Ref. 8) performed for the City of Sacramento by Dewante and Stowell, Consulting Engineers, for a residential and commercial area, North Sacramento-Hagginwood. This average design flow less the projected employee flow for 1992 provided the appropriate residential wastewater factor.

The employee waste discharge factor of 40 gpd/cap was established on the basis of extensive studies associated with the Bay Delta Study (Ref. 9) that indicated 35 gpd/cap as being representative of the general area; an additional 5 gpd/cap was added for restaurants and other complementary commercial activities in the Sacramento metropolitan area that are indiscernable in the large area from which the 35 gpd/cap derives.

During the 24-hour dry-weather flow monitoring of the State Water Resources Building that contained an employee population of approximately 3,500, it was found that the peak flow was 0.323 mgd. This provides a peak waste discharge factor of 92.5 gpd/cap. Applying the appropriate peak factor, which will be described later in this section, to the peak flow produced an average employee wastewater discharge factor of 38.5 gpd/cap, which compares very favorably with the 40 gpd/cap used in this study.

The total dry-weather sanitary flow tributary to Sump No. 2 is and will be generated in the Study Area, North Sacramento-Hagginwood, and County Sanitation District No. 2.

The sewage flow from Sanitation District No. 2 is fixed at 12 mgd by contract and is not likely to change unless a new agreement is reached between the City and the District.

Sanitary sewage flow from North Sacramento-Hagginwood was based on the 1992 projected residential and employee populations and their respective wastewater discharge factors, in addition to an anticipated peak industrial waste flow of 16 mgd.

The chemical and biological characteristics that were considered of major importance in regard to the conduct of this study were the total suspended solids, biochemical oxygen demand (BOD) and fecal coliform concentrations. Other important wastewater constituents, such as oil and grease, floatable materials and debris, and settleable solids, were not considered because of insufficient data on their concentration in the receiving waters and in the storm water runoff. However, the control of the selected wastewater quality parameters by the treatment processes employed, i. e., dissolved air flotation, mechanical screening, and retention basins, will certainly effect major removals of these pollutants.

Design sanitary sewage characteristics are shown in Figure 21 which depicts the anticipated hourly variations in flow rate and total suspended solids, BOD, and fecal coliform concentrations. These values were based on the temporal form of the results reported (Ref. 10) by the City and County of San Francisco for a similar drainage area in that City. Flow rate results were adjusted to better represent the inflow conditions expected at Sump No. 2, which is the common conveyance terminus of the Study Area, and concentration magnitudes were adjusted in proportion to the reported 24-hour averages for the two areas. Subsequent field sampling and analysis in the Study Area established that in general the design peak concentrations of BOD and total suspended solids were about 35 percent lower than the observed and the design minimum concentrations were about 45 percent lower. Design peak fecal coliform concentrations were 130 percent higher, while the design minimum was 200 percent higher. A single diurnal peak concentration of both BOD and suspended solids was observed to occur at around 3:00 p. m., which upon adjustment to correspond to the arrival time at Sump 2, would place it between the two peaks assumed in the design. The observed fecal coliform concentration maximum, however, occurred around midnight instead of noon, as was assumed in the design (cf. Figure 16).

Storm Runoff

Design flow characteristics of storm water runoff from the Study Area were based either on the application of the unit hydrograph developed for the Study Area to eighteen continuous years of hourly rainfall records or the use of a particular modification of the rational method using rainfall intensity-duration-frequency relations established in a recent hydrologic investigation (Ref. 11) for Sacramento County by Nolte Consulting Civil Engineers, Inc.

The rational method was used to determine peak storm water runoff flows in each of the individual pipes comprising the collection and conveyance system. Maximum runoff was computed by the equation $Q = CAI$, where Q is flow in cfs, C is runoff coefficient, A is contributing area in acres, and I is rainfall intensity in inches/hour for

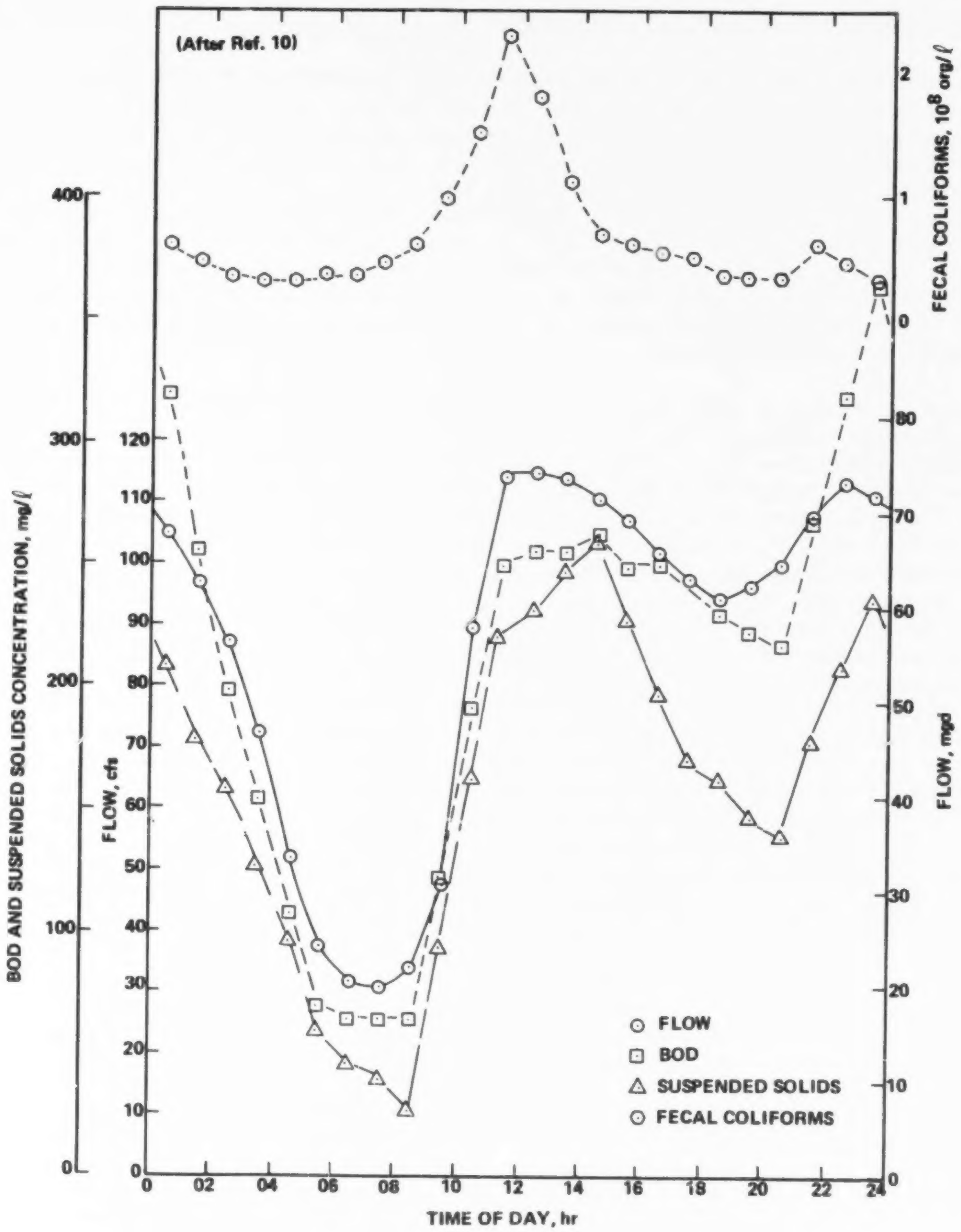

Figure 21. DESIGN SANITARY SEWAGE CHARACTERISTICS

a time duration equal to the time of concentration. Several storm recurrence intervals were considered in this study. For economic reasons, the greatest recurrence interval chosen for this study was ten years. However to obtain perhaps a more realistic perspective for the conveyance and treatment design effort, the 5- and 2-year storm recurrence intervals were considered also.

The runoff coefficient expresses the fraction of rainfall that appears as runoff after equilibrium conditions in the drainage area have been reached and is dependent upon the nature of the drainage surface. They were assigned to the individual tributary areas of the Study Area on the basis of proposed future zoning established by the Sacramento City Planning Department. Table 5 presents the runoff coefficients used in this study, based on the general character of the tributary area.

Table 5

DESIGN RUNOFF COEFFICIENTS

Description of Area	Runoff Coefficient
High-density commercial	0.80
High-density industrial	0.75
Medium-density commercial and industrial	0.60
High-density apartments	0.70
Medium-density apartments	0.60
Low-density apartments	0.50
Single-family residential	0.40
Schools	0.30
Parks and cemeteries	0.20

For the purposes of storage, treatment, and disposal design and analysis, storm water runoff characteristics were established on a real-time hourly basis from a continuous 18-year record of hourly rainfall extending from June 1950 through May 1968, and a storm water runoff quality model developed in this program. A hydrograph, developed using the observed runoff record from three more or less isolated storms, was applied to hourly precipitation data to obtain expected flow distributions. Hydrograph ordinates were least-square values of storm runoff, determined from pumping records at Sump No. 2, from a 1-in., 1-hour rainfall in the Combined Area for the hour of rainfall and the four succeeding hours. The computed ordinates were increased by a factor to reflect the higher average Combined Area runoff coefficient of 0.534 expected to pertain in the year 1992. The present average runoff coefficient was found to be 0.362.

A unit hydrograph for all storm water runoff originating in the Separated Area and converging to a common central location was prepared by modifying the unit hydrograph developed for the Combined Area. Both areas have similar characteristics and provide approximately the same maximum travel times for flow based on a 2-year storm design of about 170 minutes, so that the ordinates of the Combined Area unit hydrograph could be adjusted simply by proportioning with the ratios of the respective areas and average runoff coefficients. The area of the Separated Area draining to the common point would be 3,632 acres and the average runoff coefficient for this area was calculated to be 0.438.

For collection of all storm water runoff produced in the total Study Area at Sump No. 2, a unit hydrograph was prepared that represented inflow to Sump No. 2 from the combination of the Combined Area unit hydrograph and the Separated Area unit hydrograph modified to reflect the time of transport from the common collection point to Sump No. 2. On the basis that the time of concentration would increase by 60 minutes to 230 minutes, the base of the Separated Area unit hydrograph was expanded by 35 percent and the ordinate was proportionately diminished to provide the same total amount of runoff from an equal amount of rainfall. Table 6 provides the ordinates of the unit hydrographs.

Table 6

DESIGN UNIT HYDROGRAPHS

	Average Runoff, cfs		
Hours after Rainfall	Combined Area	Separated Area	Total Area
0.5	187	79	230
1.5	1178	499	1337
2.5	1102	467	1471
3.5	825	350	1167
4.5	507	215	785
5.5	0	0	205
6.5	0	0	114

The content of total suspended solids, BOD, and fecal coliform bacteria in storm runoff was determined on the basis of a model which depicts a constant rate of pollutant deposition on the land surface. There the pollutant decays or is removed at a constant percentage rate, i. e., as a first-order reaction. During dry weather the accumulation of pollutant on the land will therefore approach an asymptotic level. In rainy weather, however, the pollutant washes off at a rate proportional both to the amount accumulated and to the runoff flow. Heaviest concentrations occur near the beginning of storms when the land accumulation is greatest. Symbolically, the mass balance of pollutant on the land surface is $\Delta L = P - k_1 L - k_2 LQ$, where L is accumulation of pollutant over the Combined Area, ΔL is hourly increase (or decrease) in pollutant accumulation, P is production rate of pollutant, k_1 is decay constant, k_2 is wash-off constant, and Q is storm water runoff flow. The mass flow of pollutant in the runoff is $k_2 LQ$ and its concentration, $K_2 L$. The model involves three parameters, P, k_1, and k_2, whose values were selected in accordance with storm water runoff data taken in Sacramento and elsewhere (Refs. 3, 10, 12). Table 7 gives the values used for each of the three critical pollutants.

Table 7

DESIGN CONSTANTS FOR DETERMINATION OF STORM WATER RUNOFF QUALITY

Parameter	BOD	Total Suspended Solids	Fecal Coliforms
Production Rate (P), mg/hr; org/hr	3×10^{8}	6×10^{8}	6×10^{11}
Decay Constant (k_1), 1/hr	6×10^{-3}	6×10^{-3}	4×10^{-2}
Wash-off Constant (k_2), 1/(cfs)(hr)	7×10^{-4}	7×10^{-4}	7×10^{-4}

Combined Sewage

Projected combined sewage characteristics were determined by admixing the sanitary sewage and storm water runoff as ascertained by the methods described above. Seasonal industrial wastewater flows from the food processing industry were excluded from the combined sewage because these flows would not be expected to occur simultaneously with the large storm flows that occur only in the winter months in the Sacramento area.

CONVEYANCE CONSIDERATIONS

For the design of the conveyance systems it was assumed that only a minor infiltration would occur with very little effect on pipe capacity design. Investigation showed that the groundwater level is well below most of the existing sewer except in very close proximity of the Sacramento and American Rivers where high groundwater can be encountered when River flows are high. With the use of newly available pipe joints, proper inspection, and maintenance of high construction standards, it was assumed that infiltration would be negligible for all new designs.

Standard pipe diameters were used throughout the design, with minimum diameters of eight inches for gravity systems and 1.5 inches for force main systems. Minimum acceptable depth of cover over gravity-flow pipes was three feet.

Hydraulic Criteria

For the gravity systems, frictional head loss was determined from Mannings formula using a roughness coefficient, n, of 0.013, which is the design criterion used by the City.

Sanitary gravity sewers were designed on the basis that depth of flow would not exceed three-fourths of the pipe diameter and that minimum slopes were those giving 2.0 fps velocity at full-flow conditions. No separated sanitary sewer pipes were permitted to be smaller than any upstream pipe, and the soffits of the pipes entering and leaving the manholes or junctions were set at the same elevation.

For the gravity systems for the collection and conveyance of storm water runoff and combined sewage, the designs were based on pipes flowing full and at a minimum velocity of 2.0 fps. At junctions, the invert of the downstream pipe was never higher than the inverts of any incoming pipe. A portion of the downstream pipe was permitted to flow under pressure to accommodate this condition.

In the sanitary sewage force main design, frictional head loss was computed using the Hazen-Williams formula and a friction coefficient, C, equal to 135 for 6-in. and smaller diameter pipes and 115 for larger pipes. Pipe size was selected to provide the largest standard diameter that would provide for a minimum scouring velocity at the design flow. These minimum velocities were determined in an independent study (Ref. 13) and are presented in Table 8.

Because of insufficient data to determine the exact capacity of each and every pipe in the existing collection and conveyance systems, it was assumed that all existing pipes had slopes corresponding to their diameters. These slopes were ascertained from a rough correlation of slope versus diameter for the few pipes in the system whose slopes were known. The relation is shown in Figure 22. The slope for the

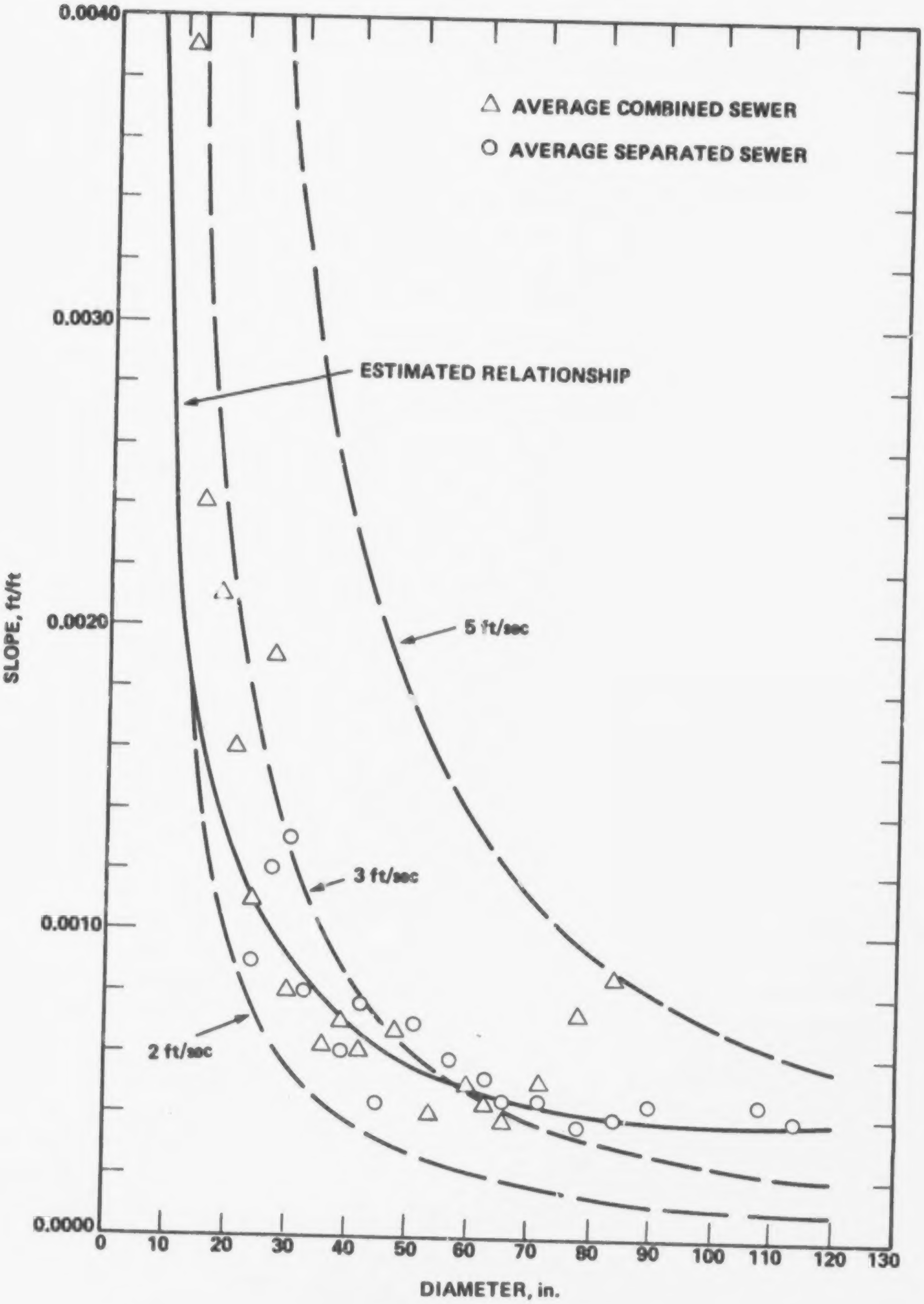

Figure 22. RELATIONSHIP BETWEEN EXISTING PIPE DIAMETER AND SLOPE

entire length of 108-in. pipe was known and constant at 0.065 percent, and thus was employed instead of the value indicated by the correlation.

Table 8

MINIMUM FORCE MAIN VELOCITIES

Diameter, in.	Velocity, fps	Diameter, in.	Velocity, fps.
1.5	0.61	24	2.45
2	0.71	27	2.59
2.5	0.78	30	2.74
3	0.86	33	2.87
4	1.00	36	3.00
6	1.22	39	3.13
8	1.42	42	3.24
10	1.58	45	3.35
12	1.72	48	3.46
14	1.86	51	3.56
16	2.00	54	3.67
18	2.12	57	3.78
21	2.29	60	3.89

Materials Selection

In order to compare alternative conveyance systems on as equal a basis as possible, the material selected for new systems was the same as those materials currently in use in Sacramento. No attempt was made to evaluate or compare these materials with others. Therefore, for example, since Amerplate is not presently in use in Sacramento, it was not specified or costed in new sanitary lines. Likewise, since corrugated metal pipe is not now in use, it was not specified for new storm sewer applications.

The materials selected for separated storm water runoff and combined sewage systems were nonreinforced concrete pipe in the 8- and 10-in. diameter sizes and reinforced concrete pipe in 12-in. and larger sizes. The pipe joints were tongue and groove except when pipe was laid in groundwater, in which case the joints were rubber gasket.

The sanitary sewage collection system material selected was vitrified clay pipe in standard sizes up to and including 42-in. diameter pipe with plasticized polyvinyl chloride or polyurethane elastomer joints.

The larger sizes, 48-in. diameter and larger, were constructed of reinforced concrete with tongue and groove joints except in ground water where joints were rubber gasket.

The reinforced concrete pipe material strength was expressed in D-load factors which vary with type of installation, depth of trench, size, and live loads imposed. The D-load is the load on the pipe per lineal foot per foot of internal diameter. The computations were based on an ideal trench section wherein the trench width at the top of pipe would not exceed the outside diameter of the pipe plus 16 inches of working space.

The material strength of nonreinforced concrete pipe and vitrified clay pipe is based on a 3-edge bearing support. Both types of pipe were utilized at their "extra strength" classification for ideal trench conditions within their limits of depth. At shallower depths a concrete cradle was provided to increase the load factor as trench width and applied loads increased.

The material selected for force mains was polyvinyl chloride for the small sizes of 1.5- through 6-in. diameters. Their strength designation was Class 125. The pipe joints were "Ring-Tite" with 0-ring seals. Larger force mains through 36-in. diameter were epoxy-lined asbestos cement pipe, Class 100. Force mains greater than 42-in. diameter were reinforced concrete pipe with rubber gasket joints.

Grinder-Pump Requirements

For the ASCE Force Main systems, the grinder-pump requirements were estimated on the basis of population density. Specifically, the number and size of grinder-pump units needed to service each major category of land use--single-family residential, multiple-family residential, low-density commercial, and high-density commercial--in the Study Area were established. Representative populations and areas used in these determinations were selected from the sewer system input design data. The derived relationship used in the design and costing of the grinder-pump components is presented in Figure 23.

STORAGE

Volumetric storage requirements for the containment of storm water runoff and wet-weather combined sewer flows were selected to match the conveyance system with regard to storm recurrence interval. Thus, a reservoir comprising a portion of a system provided with a conveyance subsystem designed for a 5-year storm would overflow or exceed its capacity with the same recurrence expectancy, that is once every five years.

Storage reservoir capacities were determined utilizing several constant withdrawal rates and hourly inflow characteristics computed

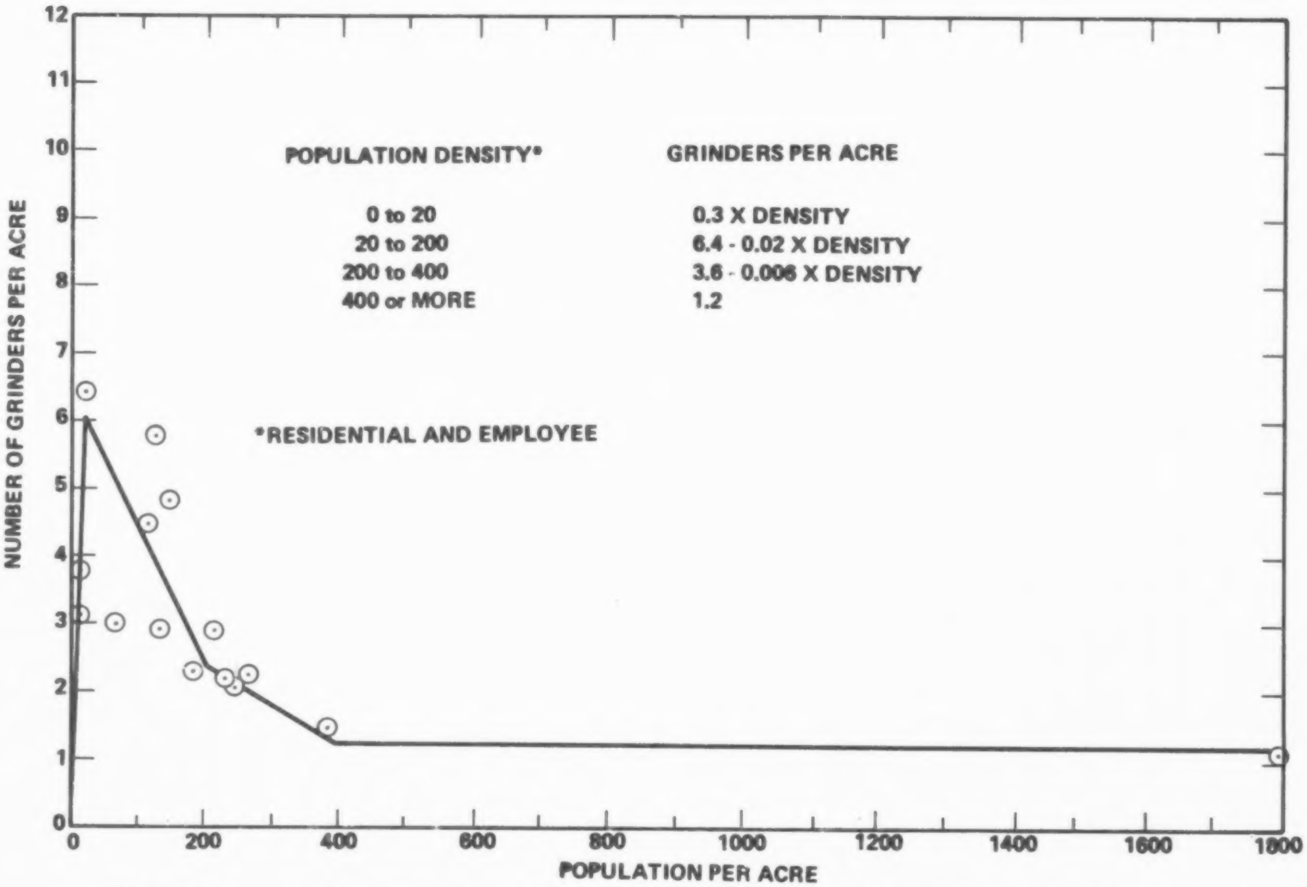

Figure 23. RELATIONSHIP BETWEEN NUMBER OF GRINDER-PUMP UNITS AND POPULATION DENSITY

from 18 years of hourly precipitation data and the unit hydrographs for the Sacramento area.

When sizing storage facilities for combined sewage, all dry-weather flow was bypassed directly to the existing municipal sewage treatment plant for processing and disposal. If any storm drainage was contained in the combined sewage, the entire flow was accepted by the pond, regardless of the extent of unused capacity available at the sewage treatment plant.

TREATMENT

To match the approximate performance capability of the mechanical screening process, the requirement for removal of suspended solids was set at 30 percent for both mechanical screening and the dissolved air flotation process. In addition, a second requirement for 60-percent removal of suspended solids was specified for dissolved air flotation only.

Dissolved Air Flotation

Pertinent design criteria established for the dissolved air flotation process effecting 30- and 60-percent removals of total suspended solids are presented in Table 9.

Table 9

DESIGN CRITERIA FOR DISSOLVED AIR FLOTATION

Parameter	Design Value 30% Removal	Design Value 60% Removal
Unit Capacity, mgd	40	40
Suspended Solids Removal, %	30	60
BOD Removal, %	21	42
Surface Loading Rate, gpd/(sq ft)	13,000	3,000
Weir Loading Rate, gpd/ft	125,000	125,000
Recirculation Rate, %	20	20
Dissolved Air Tank Detention Time, sec	60	60
Air Injection Rate, cfm at 65 psig	30	90

It was assumed that 30 percent of the total BOD contained in the storm water runoff and the combined sewage was dissolved and not removed by the dissolved air flotation treatment process. Also negligible fecal coliform bacteria removals were anticipated using this process.

Two sources of information were employed in the application of dissolved air flotation to the Sacramento study. The Rhodes Corporation, Oklahoma City, had performed testing on a pilot plant installation in Fort Smith, Arkansas, that also provided solids removal upstream by cyclone centrifugation, but only partial results were available. Therefore, design criteria were based largely on the laboratory study (Ref. 10) conducted by Engineering-Science, Inc., Arcadia, California preparatory to the design and construction of a dissolved air flotation installation on a combined sewer bypass in San Francisco. Data from this study were used to develop a relation between total suspended solids removal and surface loading rate, which is shown in Figure 24. BOD removal is also shown and was based on the assumption that 70 percent of the total BOD is in the suspended form and removed with the suspended solids.

Mechanical Screening

The mechanical screening unit employed in this study has been under development for several years by the manufacturer, SWECO, Inc. A test program is presently underway at the Hyperion Sewage Treatment Plant, Los Angeles, to determine the relationship between mesh size, flow split between treated effluent and concentrate blowdown, and pollutant reductions. These data are summarized in Table 10 which reflects both early data and also the latest available experience.

Based on the earlier results, the mechanical screening process was assumed to effect 30-percent removals of suspended solids and 21-percent reductions in BOD since it was assumed that 70 percent of the total BOD content was associated with the total suspended solids. Negligible reductions in fecal coliform density were expected through treatment of the storm water runoff and combined sewage with mechanical screening devices. At the design hydraulic loading or throughput rate at 56 gpm/(sq ft), 20 percent of the influent flow was needed for concentrate blowdown.

Stabilization Pond

Stabilization ponds, and smaller storage reservoirs, were expected to remove total suspended solids and the associated suspended BOD by sedimentation, dissolved BOD by providing time for its reduction, and fecal coliforms by permitting die-off. Total suspended solids removals were assumed to vary linearly with pond volume from no reduction at zero volume to a maximum of 70-percent removal at full capacity. Since the insoluble fraction of BOD was taken as 70 percent, the maximum removal by sedimentation would be 49 percent. The remaining

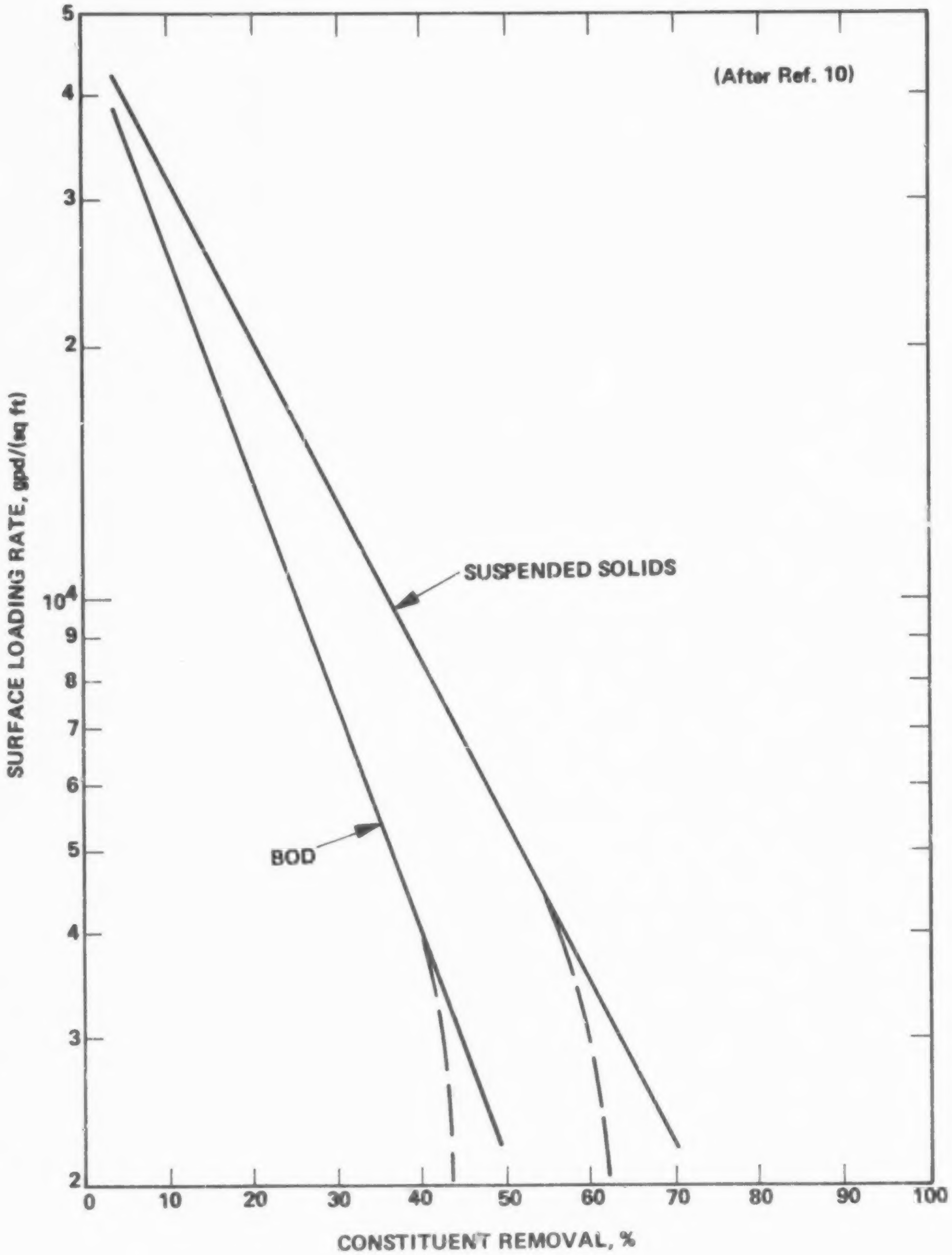

Figure 24. RELATIONSHIP BETWEEN SURFACE LOADING RATE AND CONSTITUENT REMOVAL FOR DISSOLVED AIR FLOTATION

Table 10

MECHANICAL SCREENING TEST DATA[a]

	Screen Characteristics					Average Reductions, %	
Mesh Designation	Opening, in.	Wire Diameter, in.	Open Area, %	Loading Rate,[b] gpm/(sq ft)	Treated Effluent Recovery, %	Suspended Solids	BOD
80MG	0.0070	0.0055	31.4	56	80	25.7	30.0
105TBC	0.0065	0.0030	46.9	56	80	32.0	19.0
165TBC	0.0042	0.0019	47.1	56	80	36.3	18.3
				76	88	31.7	12.0
230TBC	0.0029	0.0014	46.0	56	80	46.2	14.0
				67	82	36.2	22.7

[a] Determined on raw sanitary sewage at Hyperion Treatment Plant, Los Angeles, California by SWECO, Inc.

[b] Based on treated effluent flow

BOD, both suspended and dissolved, would be reduced at a rate of 1.0 percent each hour. A 4.0-percent hourly die-off was applied to reduce the stored fecal coliform densities.

Major reductions in fecal coliform content of the storm water runoff and combined sewage to acceptable discharge levels were expected by chlorination and retention to provide adequate contact time. At least 15-minute contact was provided between the wastewater and chlorine prior to discharge to the receiving waters, except for the systems where no storage pond was provided. In this case, only 10-minute contact was provided for the peak flow condition.

DISPOSAL

Only the disposal of wastewaters produced from storm drainage, either as separated storm water runoff or as combined sewage, was considered in this study. When analyzing combined sewer systems, it was assumed that capacity, which would be highly variable and time dependent, in the conventional sewage treatment facilities was not available. An exception was the acceptance on occasion of solids concentrate from the mechanical screening treatment process by either the existing or expanded sewage treatment plant.

The effects of wastewater discharges to the Sacramento River from the existing or future municipal sewage treatment plants were outside the scope of this program and were not included in the analysis. Obviously in evaluating downstream water quality and the total effects thereon, all discharges from the total watershed area must be considered. For this study, the quantity and composition of all storm water runoff as combined sewer overflows from the Combined Area portion of the Study Area were determined and the effects on the receiving waters immediately upon assimilation of these wastewaters subsequent to various means of containment and treatment were evaluated. The consequences of the already separated storm water discharges from the Separated Area were not included in the analysis; the pollutant loadings from these drainage areas were presumably already integrated into and contributary to the background levels used for analysis since their discharge was upstream of the background datum location.

For design purposes, three water quality parameters were assumed to adequately describe the performance of storm water runoff and combined sewage overflow water pollution control systems--total suspended solids, BOD, and fecal coliforms concentrations. Indigenous background levels in the Sacramento River were assumed to be a function only of River flow, or in the case of BOD, a constant of 2.0 mg/l. Because the relation between River flow and BOD was unknown, this constant value was selected as being representative and reasonably conservative. Relations between River flow and suspended solids and fecal coliforms concentrations are shown, respectively, in Figures 19 and 20 presented elsewhere in this report.

SPACIAL AND TEMPORAL CONSTRAINTS

To fully describe any one complete wastewater system applied to a given area, it became necessary to specify potential locations for storage and treatment facilities. Although a detailed investigation of potential sites was not undertaken, six locations were selected, from which four were utilized in costing out the alternative systems.

While not explored in this program, a promising potential was available for multiple land usage in combining a reservoir with other types of overhead facilities. This concept could be compared to the growing practice of utilizing air space over railroad and freeway rights-of-way for the construction of buildings. Such a project might be incorporated into urban renewal and implemented through an existing or new redevelopment agency.

The six sites are identified by name and location in Figure 25. It is recognized that one or more of the sites selected for costing of the various alternative systems may not be ideally suited for the planned usage. The intent has been to develop and compare the effectiveness of the candidate systems on as equal a basis as possible within the framework of a complete sewer system. The available options at any given location were kept open to the maximum extent feasible.

American River Park holding pond would be located adjacent to the Elvas Freeway and would provide a common collection point for the presently separated storm sewer systems within the Study Area. It is understood, however, that this land has been reserved by the City for a sanitary landfill site.

Sump No. 2 holding pond, a below-surface reservoir, would be constructed to the north of the existing Sump No. 2. The area is presently single-family residential, which would have to be cleared. The development of a landscaped surface park over the reservoir is suggested.

Sacramento Port holding pond would be located on land that is undeveloped and presently devoted to agricultural purposes to the west of Sump No. 2 across the Sacramento River. While located near Sump No. 2, this location in adjacent Yolo County is handicapped because of the cost of installing conduits under the River, which is a navigable deep-channel waterway.

South Sacramento holding pond would be situated on presently undeveloped land to the south of the City, about eight miles from Sump No. 2, and is considered promising. This location is adjacent to the existing Central County Sanitation District treatment facilities. While the present capacity of the County plant is limited, it is understood that it will eventually be enlarged to serve an area within which extensive growth is forecast. This southern area is relatively low in elevation and within a natural flood plain so that the construction of reservoirs may prove

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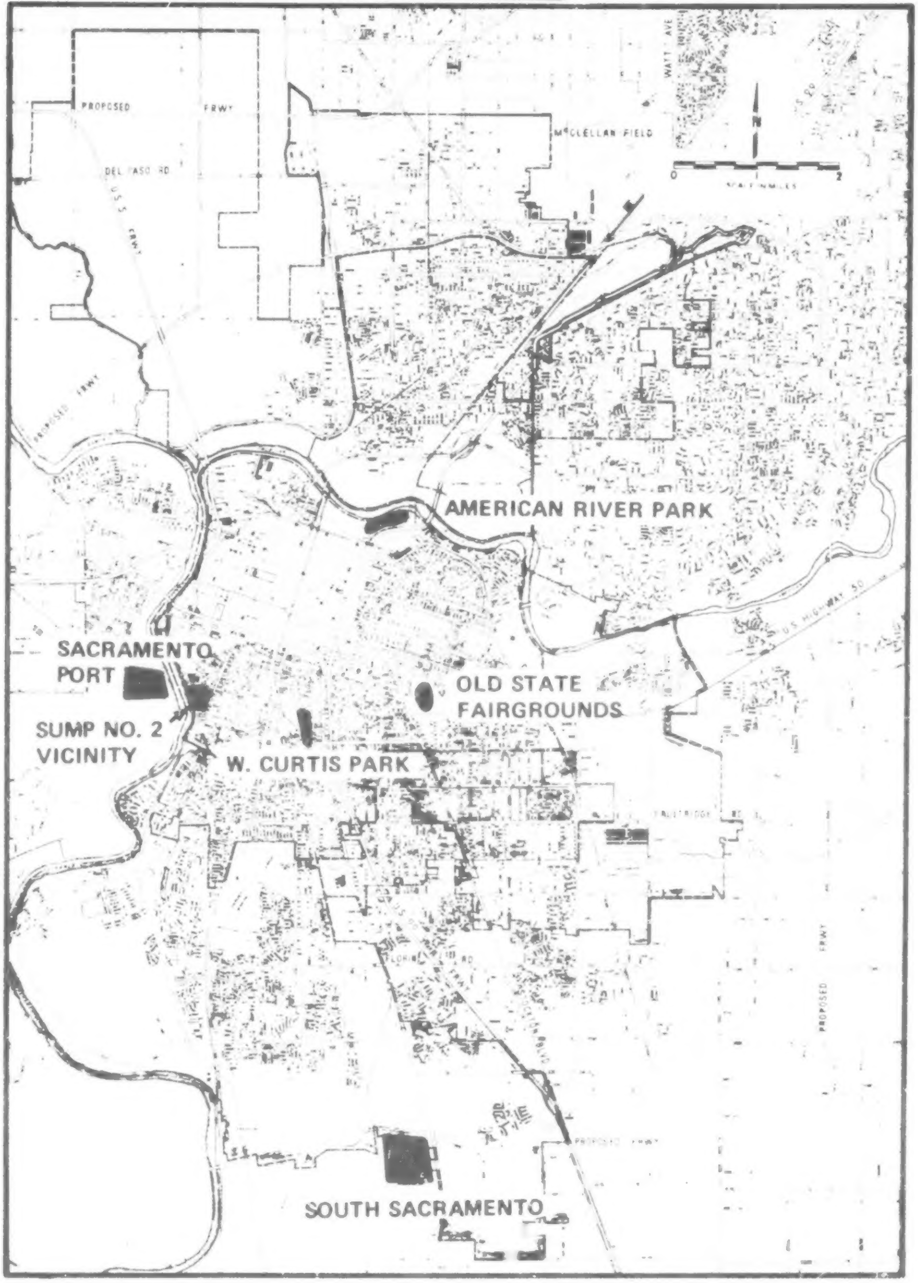

PROPOSED FRWY
DEL PASO RD
U.S. FRWY
McCLELLAN FIELD
WATT AVE
N
0
2
SCALE IN MILES
PROPOSED FRWY
AMERICAN RIVER PARK
U.S. HIGHWAY 50
SACRAMENTO PORT
OLD STATE FAIRGROUNDS
SUMP NO. 2 VICINITY
W. CURTIS PARK
FRUITRIDGE RD
FLORIN RD
PROPOSED FRWY
PROPOSED FRWY
SOUTH SACRAMENTO

Figure 25. LOCATIONS OF PROPOSED STORAGE AND TREATMENT FACILITIES

difficult.

Old State Fairgrounds holding pond would be located on the former site of the California State Fair which is unused and for sale. From an availability standpoint, a portion of this land for a reservoir location has appeal.

William Curtis Park holding pond would be situated on open land that belongs to the Western Pacific Railroad. Availability of this land is unknown however. An area between the railroad land and William Curtis Park to the east is considered a good candidate for an underground reservoir with enlarged park facilities, perhaps under the auspices of urban renewal.

PROGRAMMED DESIGN

Extensive use was made of a digital computer in three different aspects of the study. The first was design of the various sewer pipe networks--separate sanitary sewers, separate storm sewers, combined sewers, and a pressure sanitary system. Secondly, the storage requirements necessary for containment of the various quantities of flow associated with different, selected storm recurrence intervals were determined. In the third application, various proposed treatment schemes were simulated and the resulting effect on receiving water quality summarized statistically.

SEWER DESIGN

The pipe network was laid out by hand to correspond basically with the existing system. Each manhole was given an identification number. One data card was prepared for each manhole and contained the following individual information: its manhole index, index of the next manhole downstream and of any manholes immediately upstream, the tributary residential and employee populations, tributary acreage, average surface runoff coefficient, ground surface elevation, diameter and length of the existing pipe leaving the manhole, and its slope and invert elevation if known. Also given was the amount of any sewage flows pumped to the manhole from outside its immediate tributary area.

Sanitary Sewers

With the data for the entire network in the computer, the design was carried out one pipe at a time, working downstream. A search routine operated in such a way that a pipe could be selected for design only after all tributary pipes had already been designed. Tributary populations were multiplied by per capita flows to give average daily flows. Both total population and average daily flow were accumulated downstream. Design flow for any pipe was obtained by multiplying the accumulated average flow by a peak factor, which was a function of the total tributary population according to the relation,

$$\text{Peak Factor} = \frac{37.2 + \sqrt{P}}{14.3 + \sqrt{P}},$$

where P is the tributary population in 1,000's. "Exogenous" flow was also accumulated downstream and was added to the pipe design flow after application of the peak factor to the average endogenous flow.

With the design flow computed, the smallest possible pipe was selected and laid on the basis of hydraulic criteria presented previously. After completing design of all pipes, the depths and invert elevations of pipes were computed.

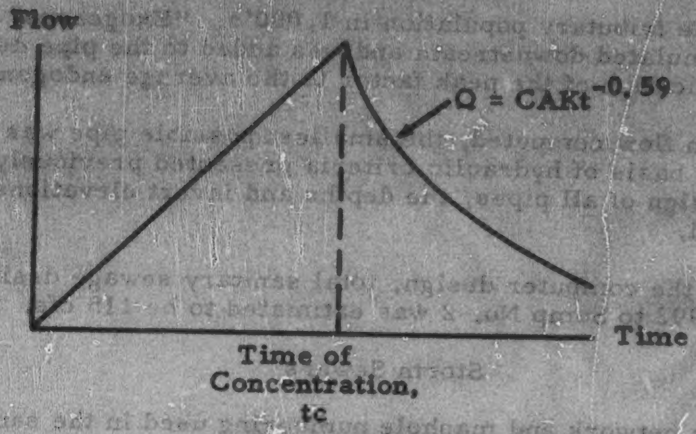
As a result of the computer design, total sanitary sewage design flow for the year 1992 to Sump No. 2 was estimated to be 115 cfs.

Storm Sewers

The basic pipe network and manhole numbering used in the sanitary sewer design was also the starting point for the storm and combined sewers. Design was carried out in the same order also, using a search procedure which assured that no pipe would be considered until after all pipes upstream had first been computed.

Flows were calculated by a particular modification of the rational methos. A uniform inlet time of 15 minutes was assumed. The intensity-duration curves (Ref. 11) had the relation $I = K t_c^{-0.59}$, where I is rainfall intensity in in./hr, t_c is time of concentration in minutes and K had the values of 4.90, 7.50, and 8.96 for the several storm recurrence intervals considered of 2, 5, and 10 years, respectively. The time of concentration and the product of runoff coefficient and area were both accumulated downstream, so that at the moment of designing the pipe below a given manhole, these two figures were available for each tributary sewer entering the manhole. The time of concentration at the lower end of any pipe was obtained from that at the upper end by adding a travel time equal to the length of run divided by velocity as computed from the design flow, slope, and diameter using the Manning formula.

It was assumed that the hydrograph at the lower end of any sewer had the form shown in the sketch on the following page. The flow was assumed to rise linearly from zero to a peak at the time of concentration. The peak flow was the cumulative tributary "CA" value (runoff coefficient times area) multiplied by rainfall intensity corresponding to the time of concentration, t_c. Thereafter the flow decreased in accordance with the intensity-duration curve. When several such hydrographs representing the branches arriving at a junction were added together, the resultant hydrograph had a series of cusp-shaped peaks occurring at the respective times of concentration for the branches. The tallest of such peaks and the corresponding time of concentration were taken as the design values for the sewer leaving.



With design flow determined, the pipe was laid at the minimum slope consistent with the previously specified criteria. After completing design of all pipes in the above manner, a second upstream-to-downstream scan was made through the system, and the upstream inverts of all pipes were lowered (if necessary) so as to lie no higher than the inverts of their tributaries. The downstream ends of adjusted pipes were also lowered, if necessary, to maintain minimum-velocity slope.

Storm water runoff design flows to each sump in the Study Area are presented in Table 11.

Combined Sewers

Gravity systems for the collection of combined storm and sanitary sewage were designed using the same method and criteria as described above for the storm sewers. Sanitary flows, computed previously and retained on magnetic tape, were merely added to the storm flows. The pipe design, and hence times of concentration, were computed using the combined flows.

Design flows in each of the four trunks at Sump No. 2 and the total for the combined sewer system at the three storm recurrence intervals are presented in Table 12.

Sewer Augmentation

A design was performed to determine the sewer construction necessary to provide sufficient capacity in the existing system for all design flows, which were taken as those previously computed for a new complete

Table 11

STORM SEWER DESIGN FLOWS

Sump No.	Area, acres	Average Run-off Coefficient	Peak Flow, cfs		
			2-yr Storm	5-yr Storm	10-yr Storm
10[a]	514	0.409	98.2	152	182
31[a]	815	0.471	151	231	277
52[a]	155	0.692	66.4	104	125
99[a]	535	0.449	110	171	205
101[a]	347	0.397	57.8	88.8	106
111[a]	414	0.750	134	206	247
A[a]	102	0.462	26.8	41.6	50.0
P[b]	445	0.200	8.0	73.5	87.8
S[b]	308	0.303	72.1	111	134
2A[c]	439	0.404	69.3	107	128
2B[c]	3,250	0.455	350	544	655
2C[c]	1,398	0.601	221	345	423
2D[c]	1,949	0.652	323	501	601
2	7,038	0.534	890	1,380	1,670

[a] Existing separated areas

[b] Proposed areas to be separated

[c] Major trunks tributary to Sump No. 2

sanitary, storm, and combined sewer system. They were compared with the capacity of the existing pipe at its assumed slope (cf. Figure 22) flowing full. Where additional capacity was required a supplementary pipe was laid at the same slope as the existing pipe, and its diameter was

Table 12

COMBINED SEWER DESIGN FLOWS

Sump	Peak Flow, cfs 2-yr Storm	 5-yr Storm	 10-yr Storm
2A*	74.2	114	137
2B*	333	496	587
2C*	257	362	422
2D*	339	512	610
2	892	1,300	1,540

*Major trunks tributary to Sump No. 2

chosen as the smallest standard size that would carry the excess flow without becoming pressurized.

Estimated flow capacities at sumps in the existing sewer systems are presented in Table 13.

Sanitary Force Main System

The force main concept involved collecting sanitary sewage at neighborhood or household stations for grinding and pumping into a network of relatively small-diameter, shallow-laid force mains. The same basic pipe layout was assumed as for the several gravity systems and design flows at any point were identical to those used in the separate gravity sanitary system. Negligible benefits would have derived in the Study Area from a different network, although in other areas that are less developed and that possess more irregular terrain, a force-main layout not constrained by gravity-flow requirements could provide benefits not evident in this study.

One half of the design flow was assigned to each of two parallel force mains, which should be necessary to assure continued operation during maintenance of the pipes at low-flow periods. Thus both parallel pipes are fully utilized at peak-flow condition. Pipe size was selected as the largest standard diameter, 1.5 inches or greater, that would provide a scouring velocity at the design flow (see Table 8).

After determination of pipe sizes, the head losses and pressure head throughout the system were computed, working upstream from the lower end. A scanning procedure was selected that provided for head computations only on pipes lying immediately upstream from another pipe whose head loss had already been established. Head at the upstream end of a pipe was obtained by adding the head loss to the downstream head and subtracting the difference in ground surface elevations.

Table 13

ESTIMATED FLOW CAPACITIES OF EXISTING SEWER SYSTEMS

Sump	Estimated Flow, cfs
10	97.5
31	73.9
52	58.2
99	127
101	106
111	86.8
A	26.2
2A*	58.2
2B*	281
2C*	318
2D*	58.2
2	716

*Major trunks to Sump No. 2

WASTEWATER QUALITY CHARACTERISTICS

To estimate the effect that the alternative systems for the control of storm water runoff and combined sewage overflows would have on the receiving water quality, a computer simulation of the wastewater generation and treatment system was developed. The model provided for calculation of storm water runoff flow and combined sewage flow during the occurrence of storm runoff, and of concentrations of pollutants in the sewage. Input data were hourly rainfall values over a continuous 18-year period of record for Sacramento, which were converted to flow and composition in accordance with criteria and techniques discussed earlier in this section.

Statistical information was extracted from each simulation and included, for each of the three pollutants, the cumulative frequency distribution of concentrations and the frequency at which particular concentration levels were exceeded.

The predicted wastewater quality distributions of total suspended solids, BOD, and fecal coliforms concentrations for storm water runoff and combined sewage from the Combined Area (excluding storm water runoff from the Separated Area) in the year 1992 are shown in Figures 26 and 27. Characteristics of sanitary sewage, which were estimated to vary diurnally only, are presented elsewhere (see Figure 21). For convenience, expected mean and maximum concentrations of the three pollutants in untreated discharges are presented in Table 14.

Table 14

PREDICTED UNTREATED WASTEWATER QUALITY CHARACTERISTICS FOR COMBINED AREA - Year 1992
(7,038 acres)

Pollutant	Wastewater		
	Sanitary Sewage	Storm Water Runoff	Combined Sewage
Total Suspended Solids, mg/l			
Mean	159	250	184
Maximum	258	693	595
BOD, mg/l			
Mean	208	125	170
Maximum	364	348	364
Fecal Coliforms, org/l			
Mean	6.8×10^7	6.3×10^4	4.1×10^7
Maximum	2.3×10^8	1.0×10^5	2.3×10^8

STORAGE CAPACITIES

To determine reservoir capacities that would be compatible with conveyance system capacities, which were dependent upon the storm recurrence interval selected for design, a computer simulation was developed. Hourly inflow to the pond was derived from the continuous 18-year hourly rainfall record using the appropriate unit hydrograph for the drainage area under consideration. For pre-selected constant withdrawal rates, the recurrence interval for the overflow or spillage from any given reservoir volume was estimated. Additional information provided by the simulation was the total duration of

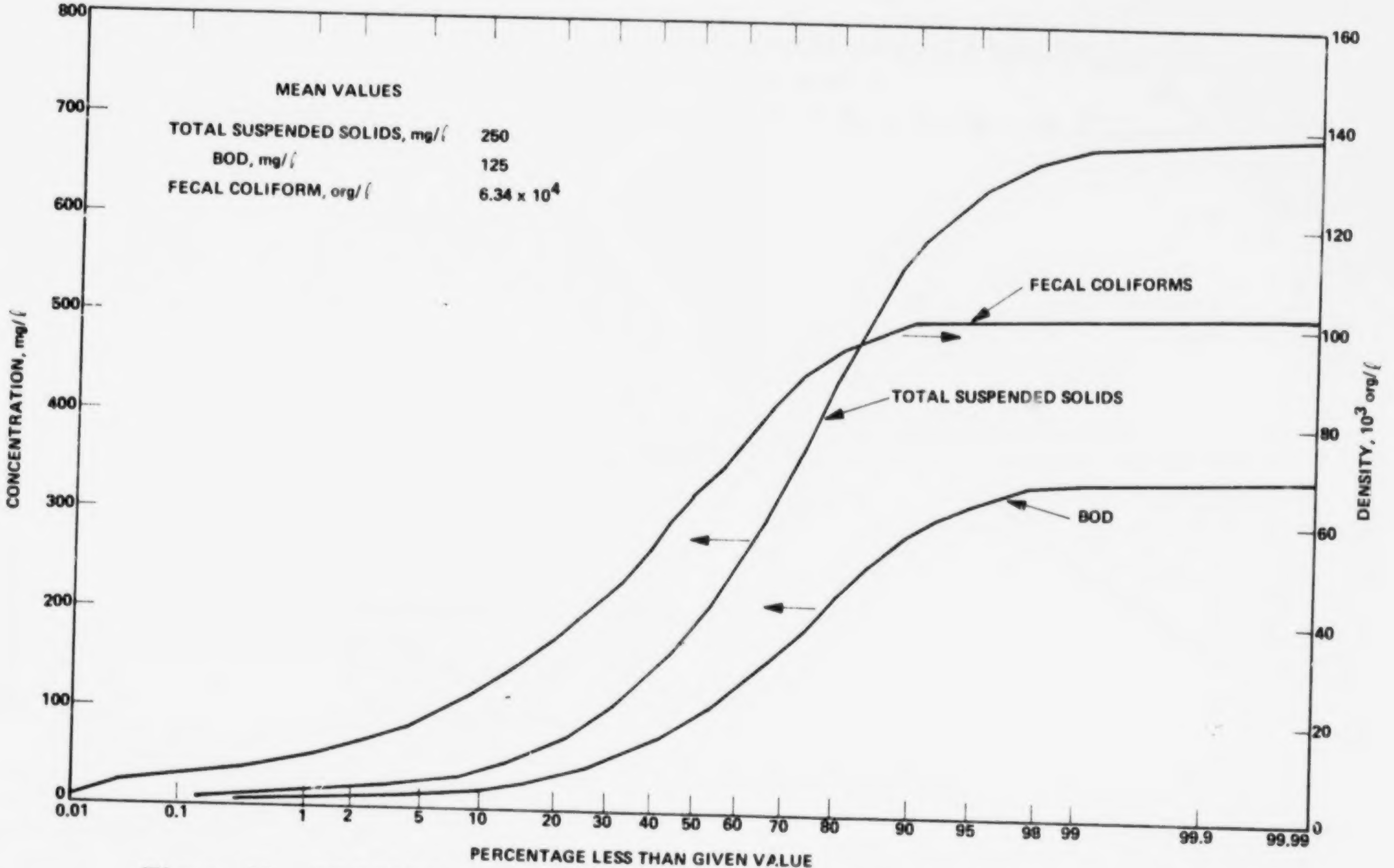


Figure 26. PREDICTED STORM WATER RUNOFF QUALITY DISTRIBUTION, COMBINED AREA, SUMP NO. 2 - YEAR 1992

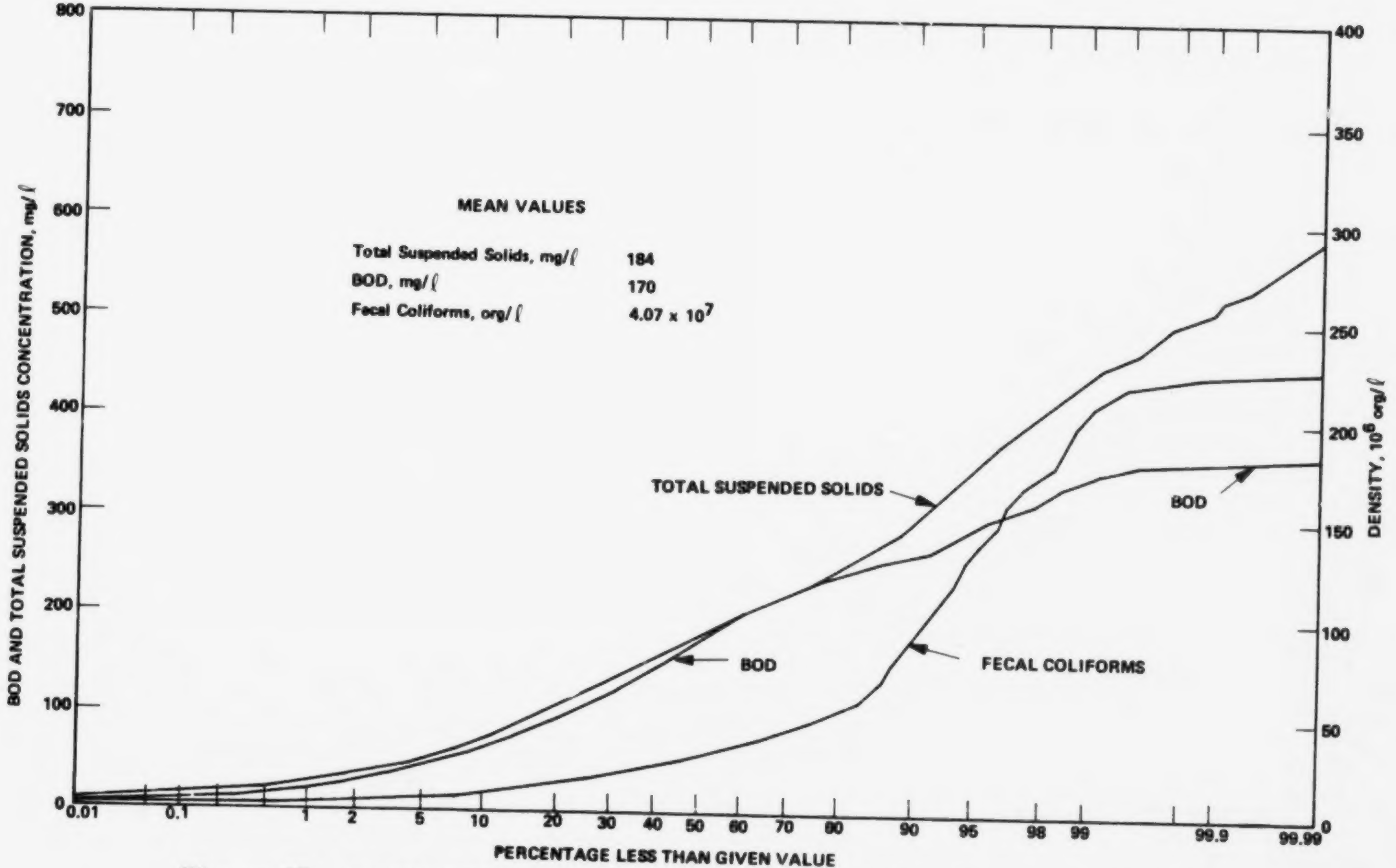


Figure 27. PREDICTED COMBINED SEWAGE QUALITY DISTRIBUTION, COMBINED AREA, SUMP NO. 2 - YEAR 1992

overflow.

The calculated relations between storage reservoir volume and reservoir withdrawal rate for storm water runoff and combined sewage flows from the Combined Area draining to Sump No. 2, for storm water runoff from the Separated Area draining to American River Park, and storm water runoff and combined sewage flows from the Total Area draining to Sump No. 2 are shown in Figures 28 through 32, respectively. It was assumed that the hydrograph for the Total Area drainage at Sump No. 2 would be applicable wherever the holding pond might be located downstream of this point since no additional flow would be introduced.

TREATMENT PROCESS PERFORMANCE

Effluent concentration distributions of the three pollutants from various sewage storage and treatment process combinations were estimated through system simulation.

The effects of treating storm water runoff from the Combined Area are shown in Figures 33, 34, and 35, which present distributions of total suspended solids, BOD, and fecal coliforms concentrations, respectively. The volumes noted refer to storage pond capacity and the removal percentages to total suspended solids reductions in that portion of the flow received by the treatment process. Storage pond overflows were not subjected to treatment.

Predicted mean and maximum concentrations corresponding to the distributions are given in Table 15. These results indicate that large holding ponds would reduce markedly the average effluent pollutant concentration without appreciable change (in this case zero) in the maximum expected value. Processes that are capable of a fixed percentage removal of from 30 to 60 percent on the other hand would effect moderate reductions in average effluent pollutant concentrations and sizeable decreases in maximum predicted concentrations.

Distributions of the three pollutant concentrations in the effluent from various storage and treatment combinations processing combined sewage overflows from the Combined Area are shown in Figures 36, 37, and 38; and the corresponding expected mean and maximum concentrations are given in Table 16. Sanitary sewage flows that occur during periods when storm runoff is nonexistant would not contribute to this wastewater quality distribution and thus were excluded from the computation.

In contrast to the storm water runoff distributions, the singular influence of a large holding pond on the attenuation of mean pollutant concentration in comparison to the combined effects of small or moderate storage facilities and fixed removal treatment processes

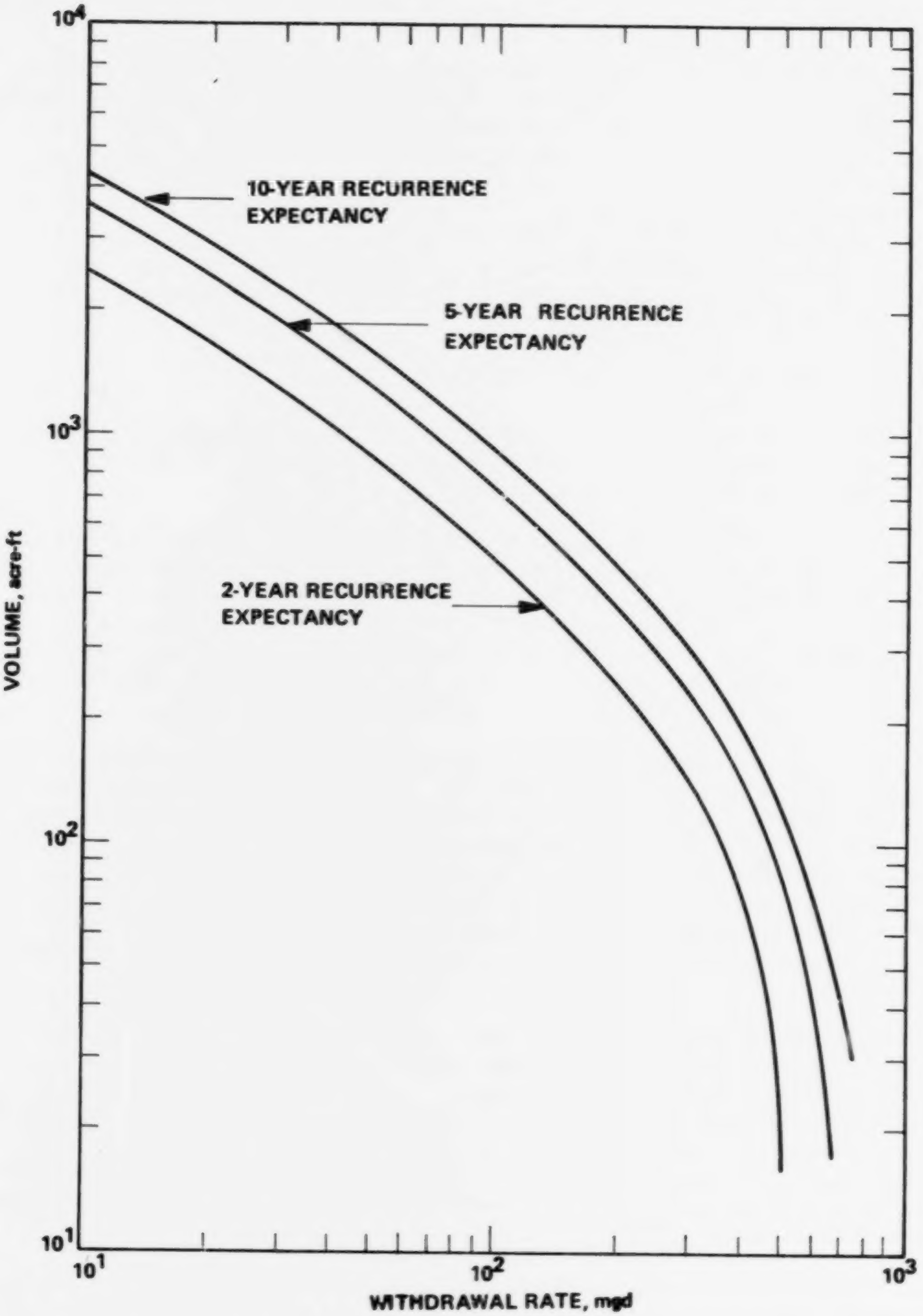


Figure 28. RELATIONSHIP BETWEEN RESERVOIR VOLUME AND RESERVOIR WITHDRAWAL RATE FOR STORM WATER RUNOFF, COMBINED AREA

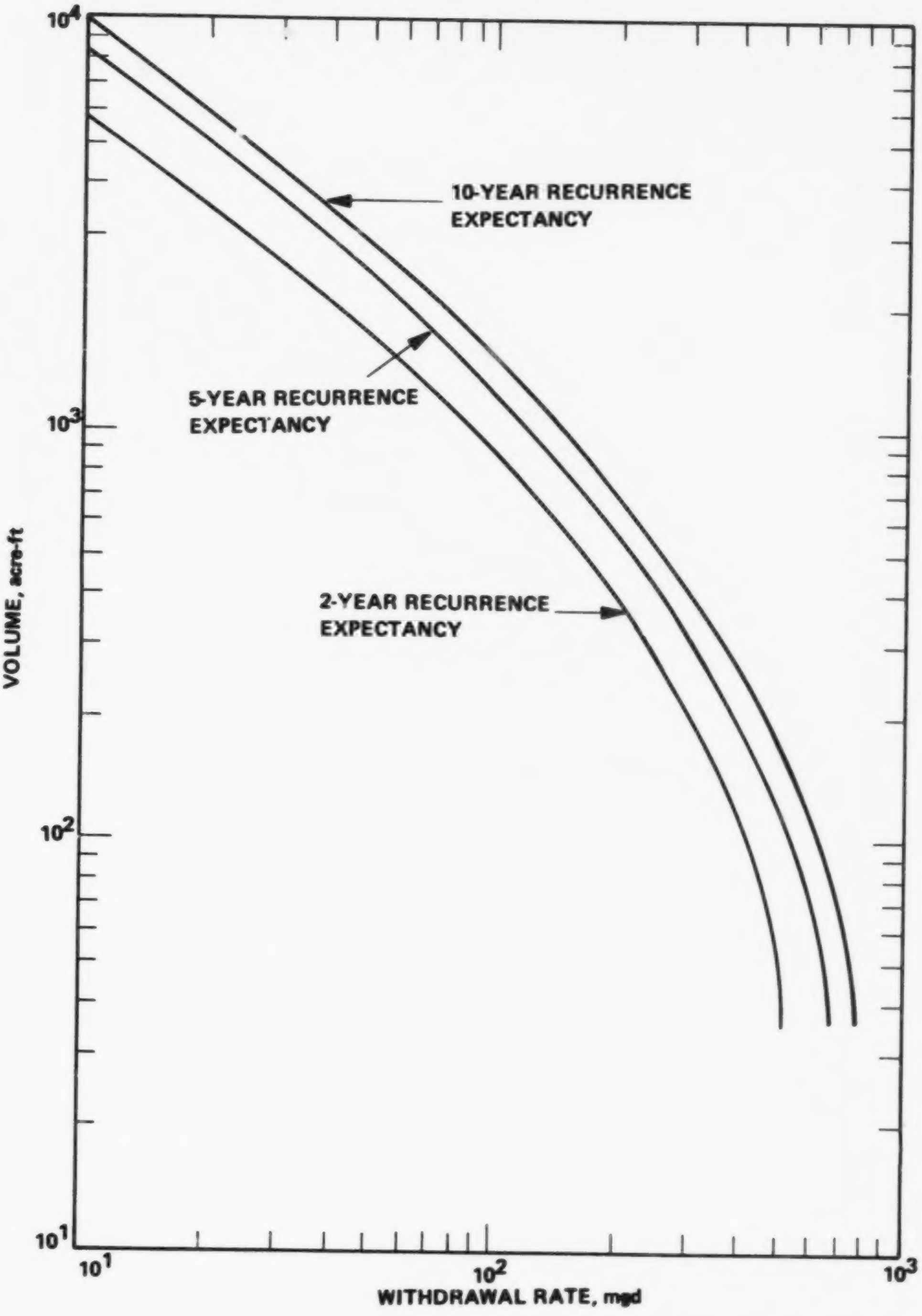


Figure 29. RELATIONSHIP BETWEEN RESERVOIR VOLUME AND RESERVOIR WITHDRAWAL RATE FOR COMBINED SEWAGE, COMBINED AREA

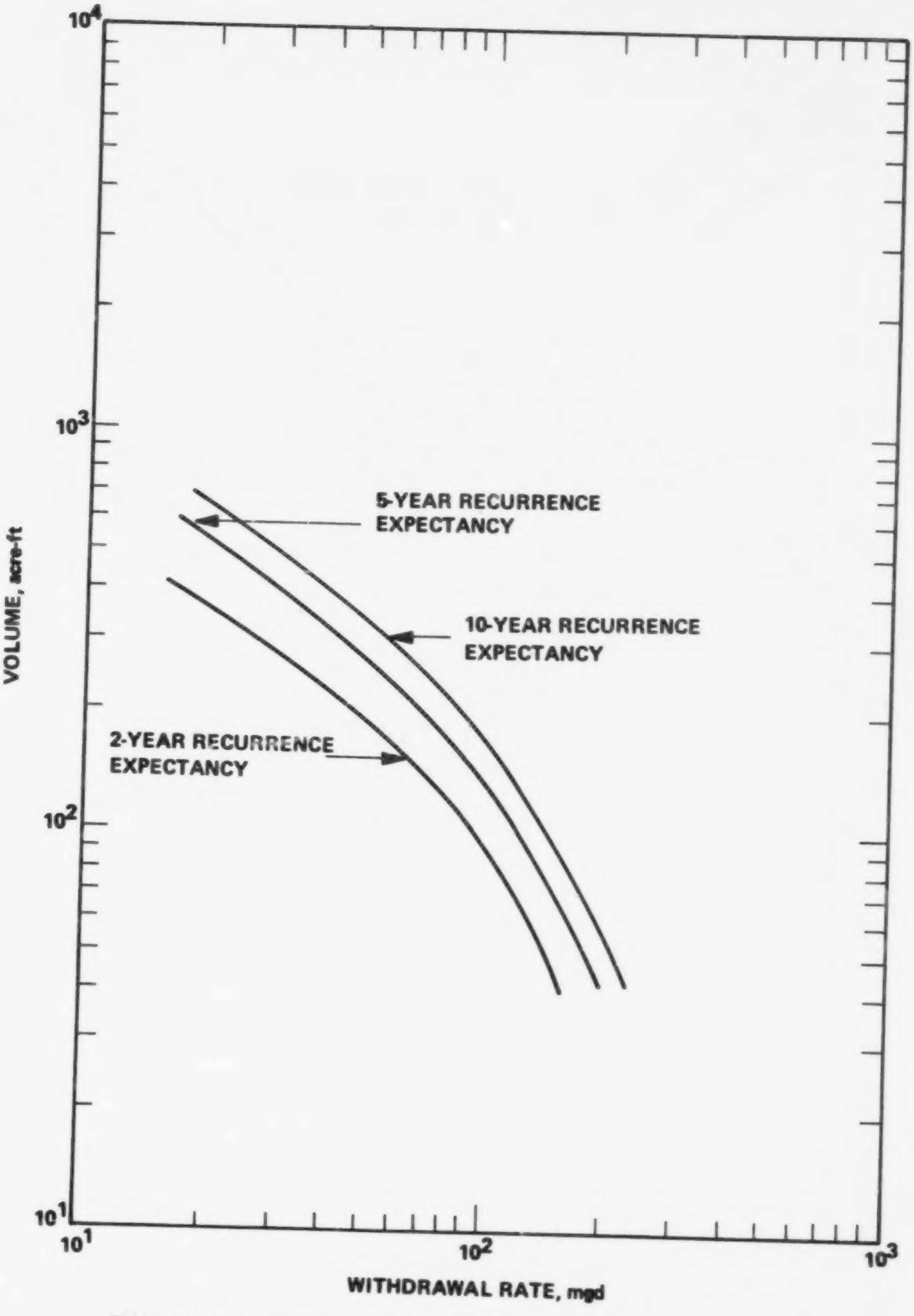


Figure 30. RELATIONSHIP BETWEEN RESERVOIR VOLUME AND RESERVOIR WITHDRAWAL RATE FOR STORM WATER RUNOFF, SEPARATED AREA

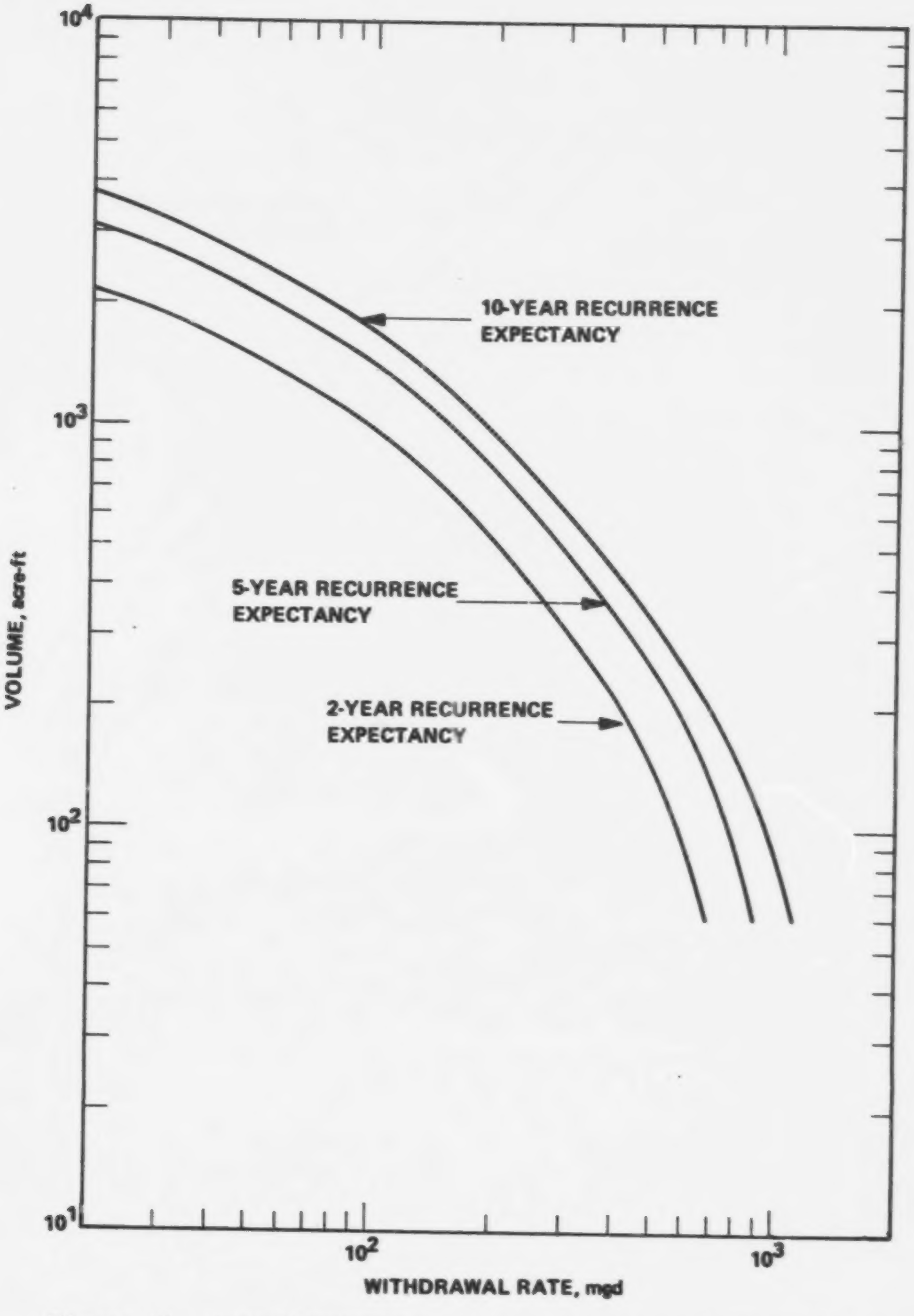


Figure 31. RELATIONSHIP BETWEEN RESERVOIR VOLUME AND RESERVOIR WITHDRAWAL RATE FOR FOR STORM WATER RUNOFF, TOTAL AREA

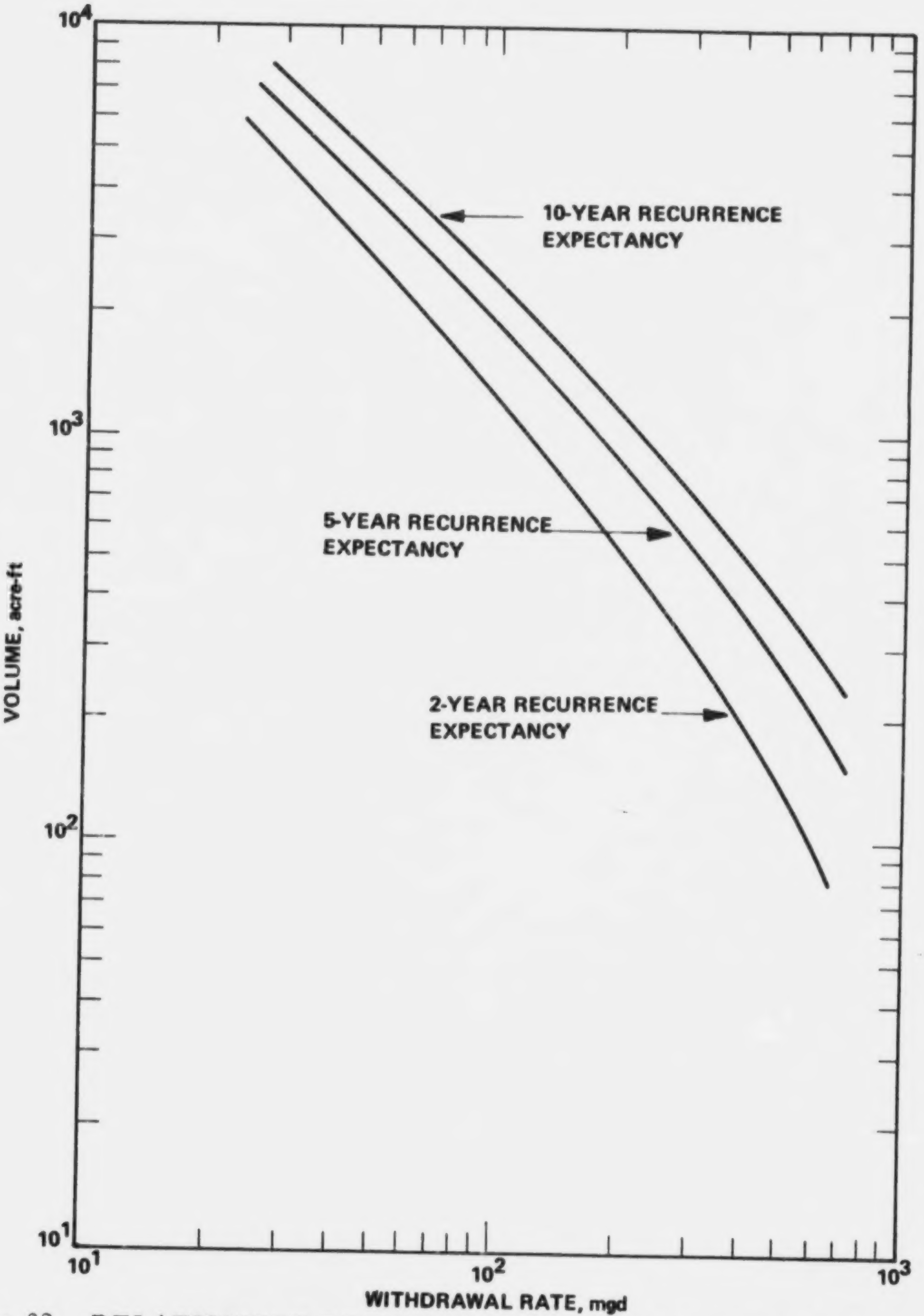


Figure 32. RELATIONSHIP BETWEEN RESERVOIR VOLUME AND RESERVOIR WITHDRAWAL RATE FOR COMBINED SEWAGE, TOTAL AREA

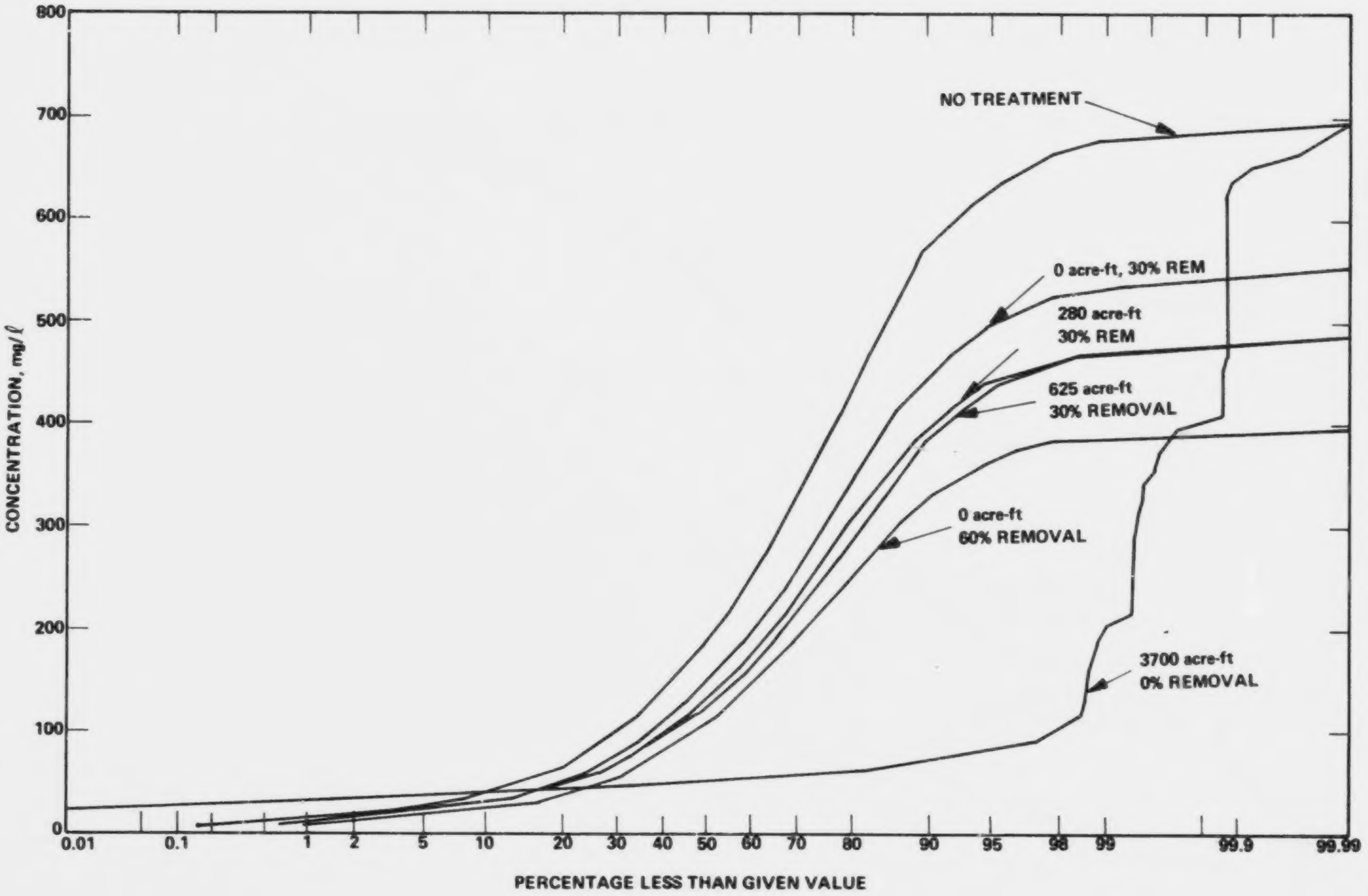


Figure 33. EFFECT OF VARIOUS TREATMENTS ON STORM WATER RUNOFF TOTAL SUSPENDED SOLIDS CONTENT, COMBINED AREA - YEAR 1992

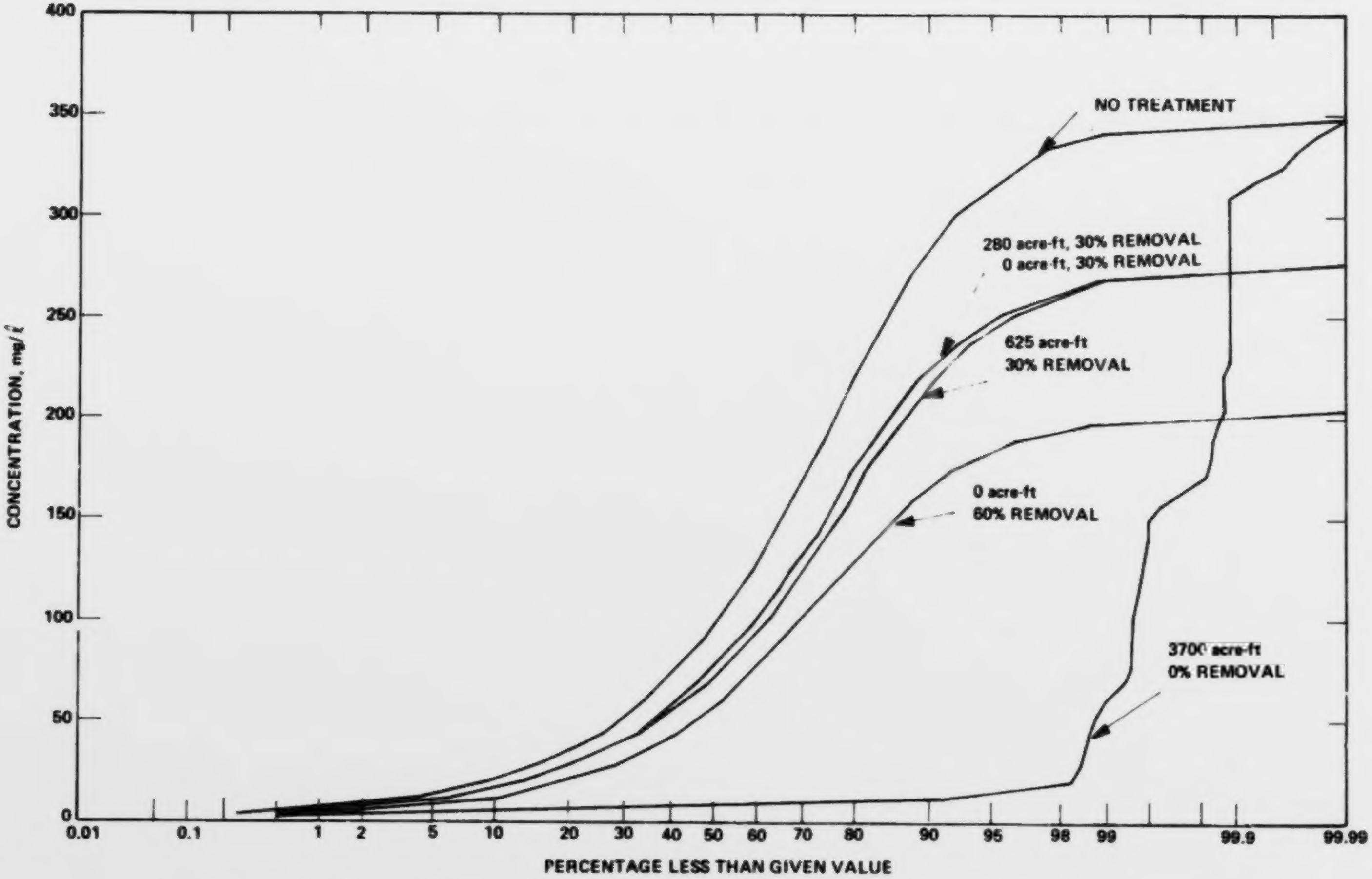


Figure 34. EFFECT OF VARIOUS TREATMENTS ON STORM WATER RUNOFF BOD CONTENT, COMBINED AREA - YEAR 1992

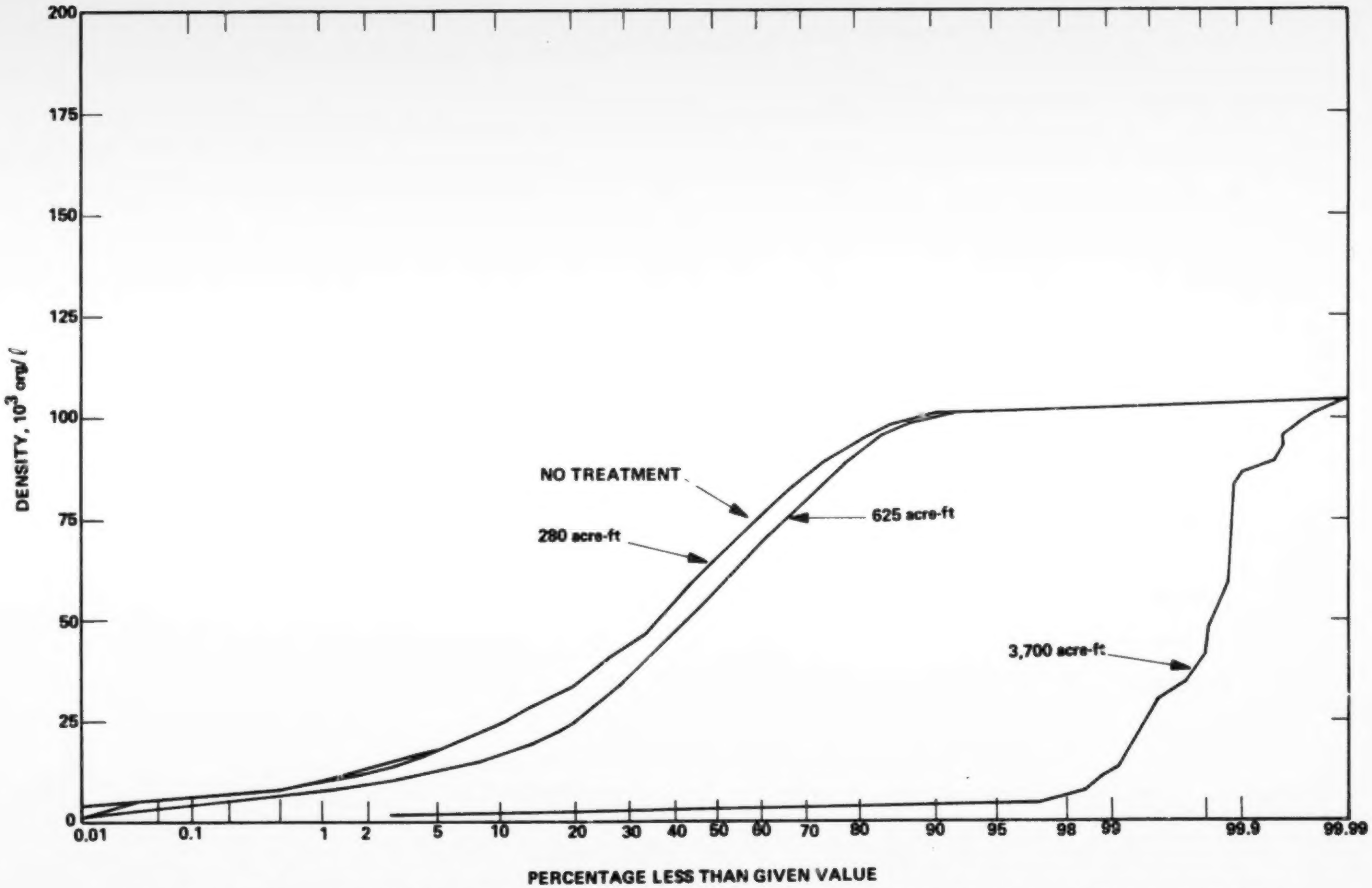


Figure 35. EFFECT OF VARIOUS TREATMENTS ON STORM WATER RUNOFF FECAL COLIFORMS CONTENT, COMBINED AREA - YEAR 1992

Table 15

PREDICTED TREATED STORM WATER RUNOFF QUALITY CHARACTERISTICS YEAR 1992

Treatment Combination	Total Suspended Solids, mg/l		BOD, mg/l		Fecal Coliforms, org/l	
	Mean	Max	Mean	Max	Mean	Max
3,700 acre-ft Pond 0% Removal	58.1	693	10.1	348	3.3×10^3	1.0×10^5
0 acre-ft Pond 30% Removal	197	553	98.6	276	6.3×10^4	1.0×10^5
280 acre-ft Pond 30% Removal	175	483	98.7	276	6.3×10^4	1.0×10^5
625 acre-ft Pond 30% Removal	168	483	94.3	276	5.8×10^4	1.0×10^5
0 acre-ft Pond 60% Removal	145	399	72.5	204	6.3×10^4	1.0×10^5
None	250	693	125	348	6.3×10^4	1.0×10^5

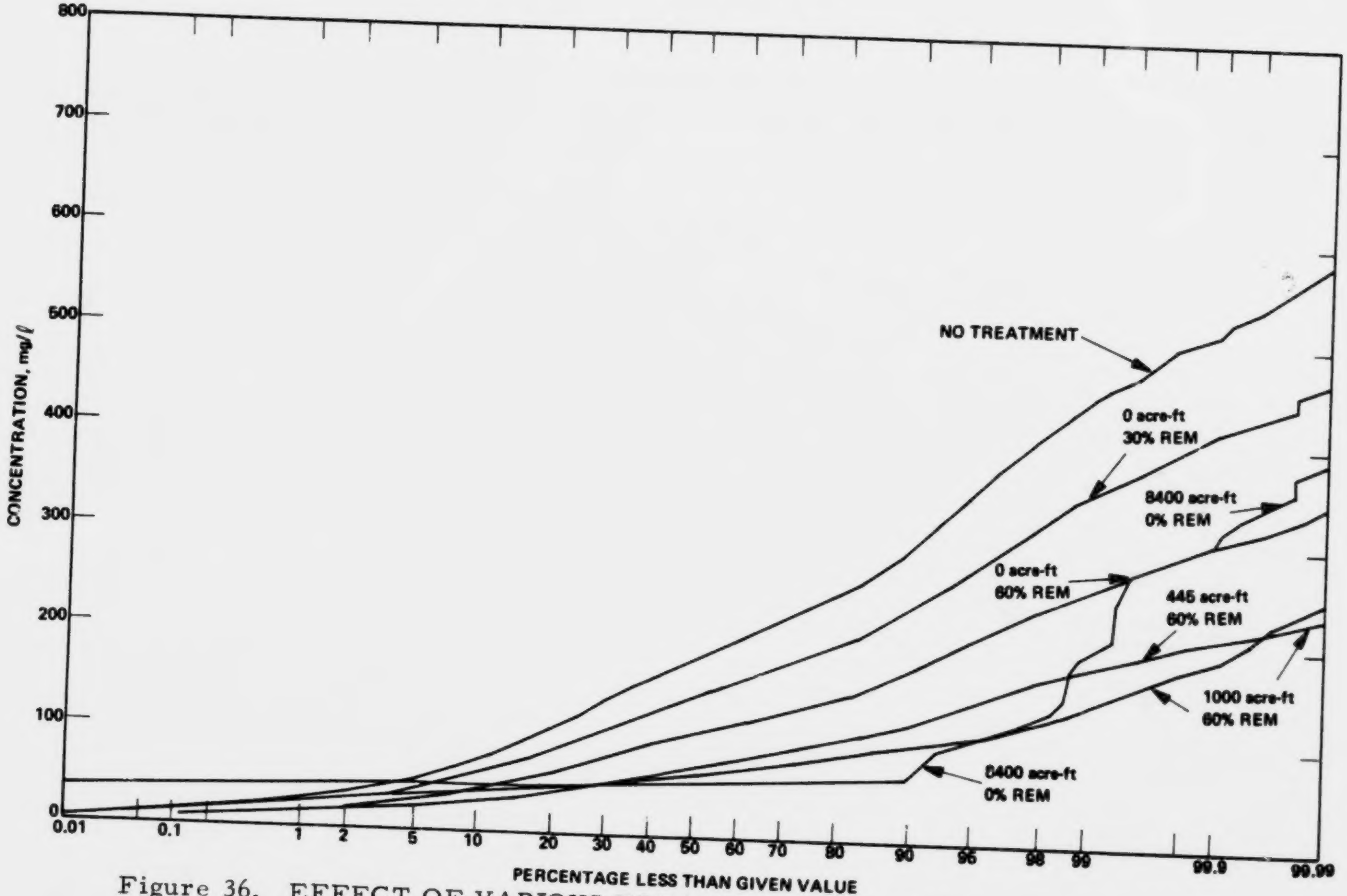


Figure 36. EFFECT OF VARIOUS TREATMENTS ON COMBINED OVERFLOWS TOTAL SUSPENDED SOLIDS CONTENT, COMBINED AREA - YEAR 1992

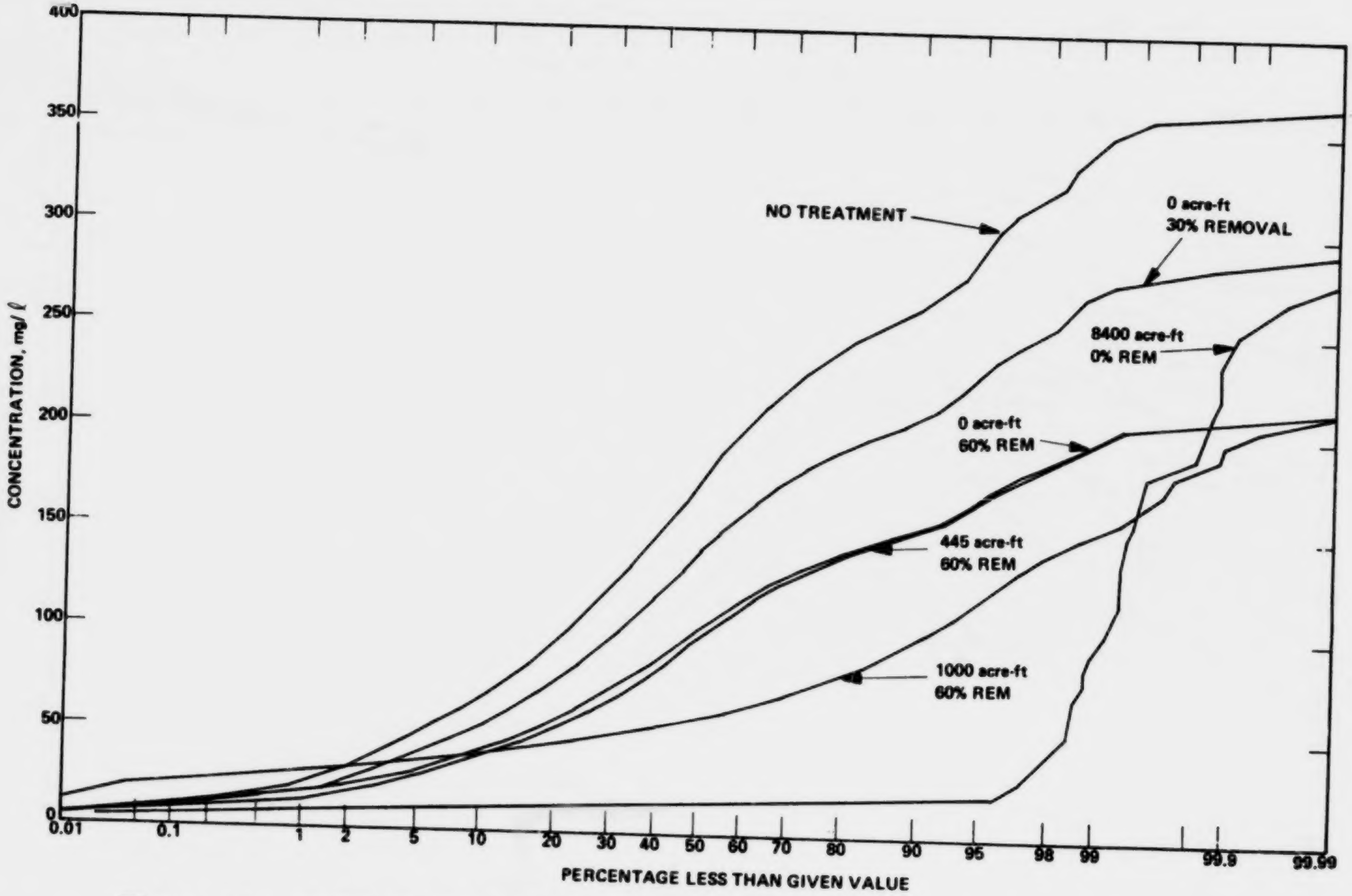


Figure 37. EFFECT OF VARIOUS TREATMENTS ON COMBINED OVERFLOWS BOD CONTENT, COMBINED AREA - YEAR 1992

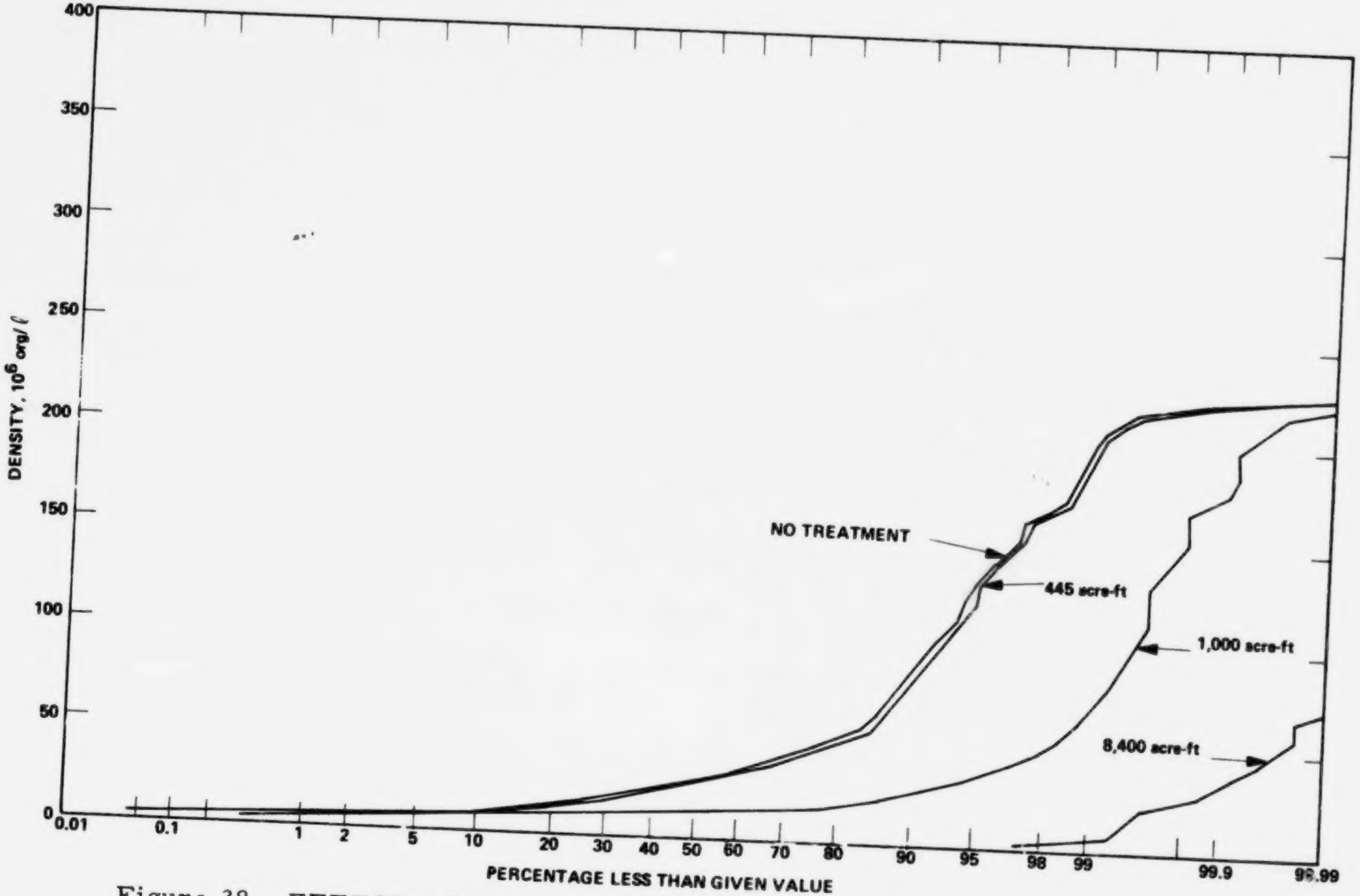


Figure 38. EFFECT OF VARIOUS TREATMENTS ON COMBINED OVERFLOWS FECAL COLIFORM CONTENT, COMBINED AREA - YEAR 1992

Table 16

PREDICTED TREATED COMBINED SEWAGE QUALITY CHARACTERISTICS
YEAR 1992

Treatment Combination	Total Suspended Solids, mg/l		BOD, mg/l		Fecal Coliforms, org/l	
	Mean	Max	Mean	Max	Mean	Max
8,400 acre-ft Pond 0% Removal	59.8	385	15.2	276	$2.6\text{x}10^{5}$	$1.1\text{x}10^{8}$
0 acre-ft Pond 30% Removal	145	483	135	292	$4.1\text{x}10^{7}$	$2.3\text{x}10^{8}$
0 acre-ft Pond 60% Removal	106	357	98.9	2.2	$4.1\text{x}10^{7}$	$2.3\text{x}10^{8}$
445 acre-ft Pond 60% Removal	73.1	245	95.8	212	$3.8\text{x}10^{7}$	$2.3\text{x}10^{8}$
1,000 acre-ft Pond 60% Removal	65.2	245	62.8	212	$1.2\text{x}10^{7}$	$2.2\text{x}10^{8}$
None	184	595	170	364	$4.1\text{x}10^{7}$	$2.3\text{x}10^{8}$

would be less. Large ponds in this case would however effect a reduction in maximum expected pollutant concentrations.

RECEIVING WATER QUALITY

It was not feasible to make use of a synthetic procedure to estimate the Sacramento River's natural pollution load. The processes governing introduction and removal of pollutants in the River are not subject to ready quantification. Instead, empirical correlations between River flow and pollutant concentration were developed from existing data. These relations are presented in an earlier subsection.

Hourly River flows were obtained by straight-line interpolation between the bracketing daily values. Concentrations of total suspended solids, BOD, and fecal coliforms were then computed as 3-point interpolations in tables expressing the aforementioned correlations.

The estimated distributions of the three pollutant concentrations in the Sacramento River for 1992 prior to wastewater discharges from the Combined Area are presented in Figure 39.

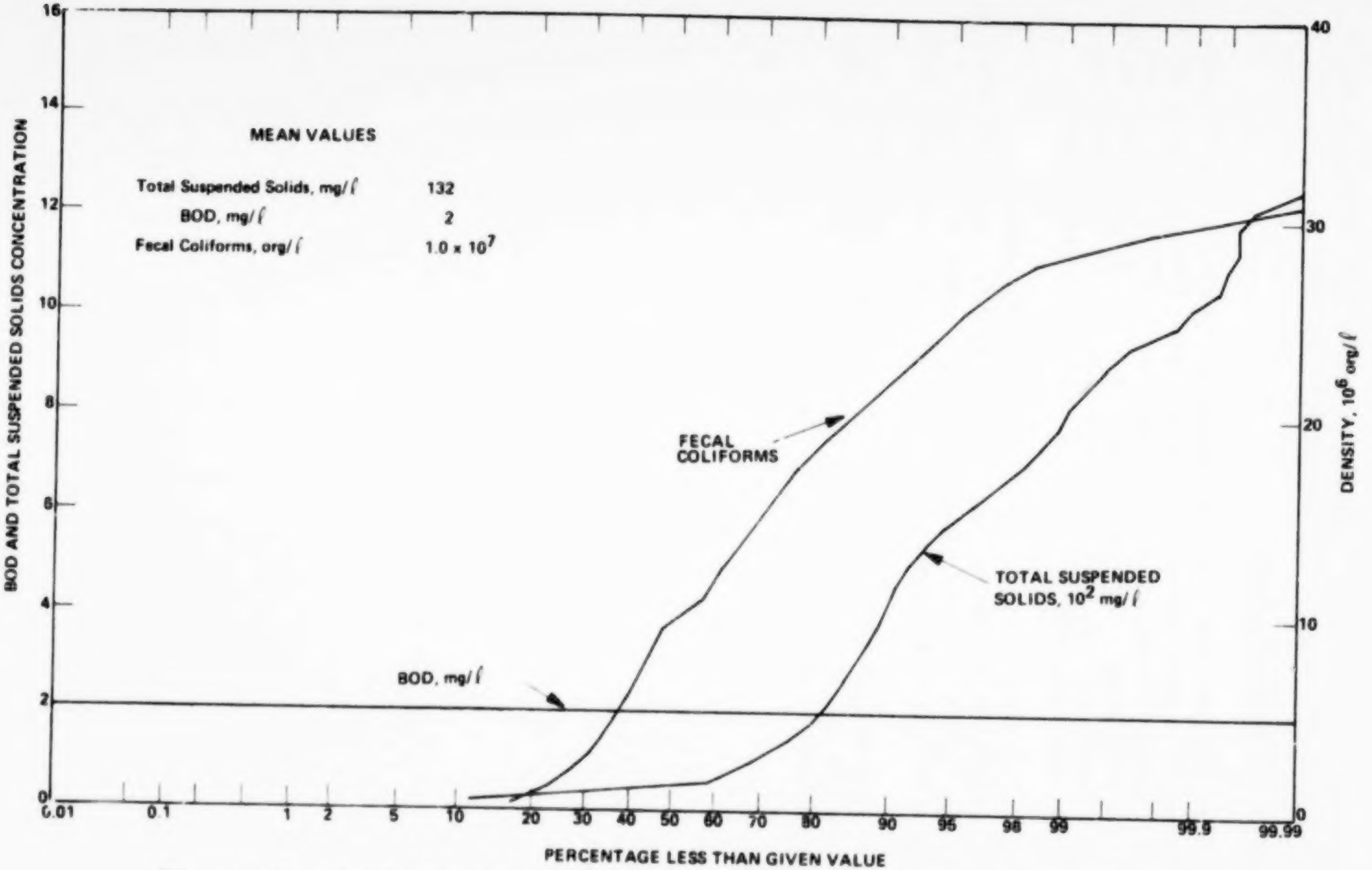


Figure 39. ESTIMATED BACKGROUND DISTRIBUTION OF POLLUTANT CONCENTRATIONS IN THE SACRAMENTO RIVER - YEAR 1992

Section VII

SYSTEMS ANALYSIS

Detailed descriptions and costs of the various components comprising the system for abatement of water pollution from storm water runoff and combined sewer overflows were prepared to permit a thorough and careful analysis and comparison of all alternatives. Although specially related to the situation in the City of Sacramento, the discussion should be construed to have general application elsewhere.

COSTING

To provide a common basis for comparing costs of the alternative systems, total annual costs for construction, installation, operation, and maintenance of each alternative were estimated.

CONSTRUCTION AND INSTALLATION

Sewer line construction cost estimating was pursued in more detail than other costing aspects of overall sewage system construction for several reasons. First, the sewer lines represented a substantial percentage of total costs. Secondly, since separation of flows was basic to several of the alternatives, the careful development of estimated costs was warranted in order to properly compare and evaluate all of the alternatives.

A general purpose computer program was developed for estimating the construction costs of complete sewer systems within the Study Area. Inputs to the costing program were received directly from the outputs of the sewer design programs simulating the various conveyance alternatives. These costs were developed for each sewer line and for each main trunk system.

For the force main concepts, the sewer line costs also include the grinder-pump costs for transfer of sewage into the force main. Sizing the number of grinder-pump units were derived from the computer programs using the relation shown in Figure 23 of the preceding section.

Construction costs of reservoirs or storage ponds varied with size and location. Curves were developed to relate cost and size over a wide range of capacities for both near-surface underground and surface reservoirs.

Some of the treatment processes under consideration had very little in the way of historical precedent for development of construction costs. In these cases, the cost backup was derived by subdividing the processes into parts for component costing. Every attempt was made to cost the alternative systems on a common basis.

OPERATIONS AND MAINTENANCE

In addition to the estimation of all capital costs for construction and installation, annual operational and maintenance costs were developed for each capital cost item. Wherever applicable, curves were developed to relate annual operational and maintenance costs to system capacity so as to cover a range of sizes where optimizing trade-off opportunities became available.

ANNUAL CONVERSION

All facilities construction costs were developed from a mid-1969 base for the Sacramento area. On all developed costs, a 15-percent upward adjustment was applied to account for miscellaneous items not covered in the initial preliminary design work. The mid-1969 costs were then converted to mid-1972 costs using an 8-percent annual escalation factor. While the 8-percent annual escalation was higher than the long-term historical experience of from four to five percent, it was lower than the 1968 annual increase of about 10 percent. Mid-1972 was selected as the target start date for new construction on the basis that this is the earliest practical start date subsequent to the necessary planning and engineering work. An allowance of 15 percent of construction cost was also applied to cover all planning, engineering design, administrative, and construction management services.

The costs of all new facilities were converted to a mid-1972 annual basis by amortizing over the life expectancy of each installation, using a 5-percent interest rate. The general guidelines established for life expectancy were perpetual life for land requirements, 40 years for sewage conveyance systems and large reservoirs, 20 years for pumping plants and treatment facilities, and ten years for selected equipment within pumping plants and treatment facilities.

The 15-percent contingency allowance and the 8-percent annual escalation adjustment were also applied to the annual operational and maintenance costs in the same manner as applied to new facilities construction costs to reach the mid-1972 construction start date. Annual escalation beyond mid-1972 for the cost of new construction, equipment replacement, and operation and maintenance was estimated at five percent.

COLLECTION AND CONVEYANCE

Two of the candidate systems entail separation of sanitary sewage and storm water runoff, thus requiring either a new storm or a new sanitary system while the existing combined system would be used to convey, respectively, the sanitary sewage or the storm water runoff. There are, in addition, other collection and conveyance options to be investigated. The boundary location between gravity separated system and force main system must be specified when considering the hybrid force main concept. Regardless of candidate system, the existing system augmentation requirements to meet a given overall criteria level must be considered. As a result, seven different collection and conveyance subsystems were analyzed.

DESCRIPTION

Gravity Separation

Three alternatives were investigated relative to complete gravity separation of sanitary and storm water flows. All three alternatives are primarily concerned with the separation requirements in the Combined Area portion of the Study Area. Each alternative subsystem contains, in addition to the basic features described below, the necessary augmentations of existing facilities to provide a complete, comparable subsystem.

New Gravity Storm Sewers (Augmented Existing System for Sanitary)

For the Combined Area, a new storm sewer installation was designed with the existing sewer system utilized for sanitary sewage only. As expected, the existing combined system was found to be more than adequate for projected sanitary sewage peak flows, with few minor exceptions, and thus requires very limited augmentation.

New Gravity Sanitary Sewers (Augmented Existing System for Storm)

For the Combined Area, a new sanitary sewer installation was designed with the existing sewer system utilized for storm water runoff only. The existing system was found to be undersized for almost its entire length, even for a storm of 2-year recurrence expectancy, and would require augmentation to provide adequate hydraulic capacity.

Augmentation of Existing Combined Sewers

For the Combined Area, the existing combined system was augmented by superimposing a new-design parallel system as required to assure that combined flow criteria requirements would be met at all points along the combined sewer length. This augmented combined system then provided a basis for evaluating the relative merits of separated versus combined flows.

ASCE Force Main Separation

The ASCE System of separation of sanitary wastewaters from storm water runoff envisions construction of individual dwelling, commercial, or industrial grinder-pump units discharging into a pressure-line system developed to convey the sanitary wastes to a treatment location. The many facets of the program ranging from consideration of individual home sewage flow variations to control and pumping techniques have been explored in detail by the ASCE (Ref. 13).

In essence, the ASCE concept proposes that each individual dwelling and building have a combination storage, grinder, and pumping facility. Where possible, the grinder-pump station would be located within the basement and the small plastic discharge piping to the common pressure line in the street would be laid within the existing service connection. There are variations in the street force mains proposed but, in principal, they are shallow lines laid in trenches in the parkway areas which proceed to a common collection point.

One of the ASCE schemes proposes the use of pressure lines within sections of existing gravity sections wherever sewers of sufficient size are available. Throughout these gravity sections, the pressure lines would be suspended from the pipe soffit on hangers, with special manholes constructed to accommodate the essential valving. Controls would be provided to maintain a positive pressure within the system to prevent alternate filling and draining of lines. The installation of the smaller diameter force mains would increase the capacity of the existing combined sewage system for use as a separate storm drainage system, thus accomplishing the separation concept.

A modification to the ASCE concept was derived in this program to eliminate most of the individual household grinder-pump units and substitute a larger zonal grinder-pump station. The merits of this scheme are economics and the potential maintenance advantage inherent with a single large station for an area rather than many small individual units. It should be noted that many existing small 8-, 10-, and 12-in. diameter sewers which have sufficient capacity for sanitary flows but not for storm runoff, requiring replacement in any separated storm sewer application, would be utilized in this approach to convey sanitary sewage to the zonal grinder-pump units.

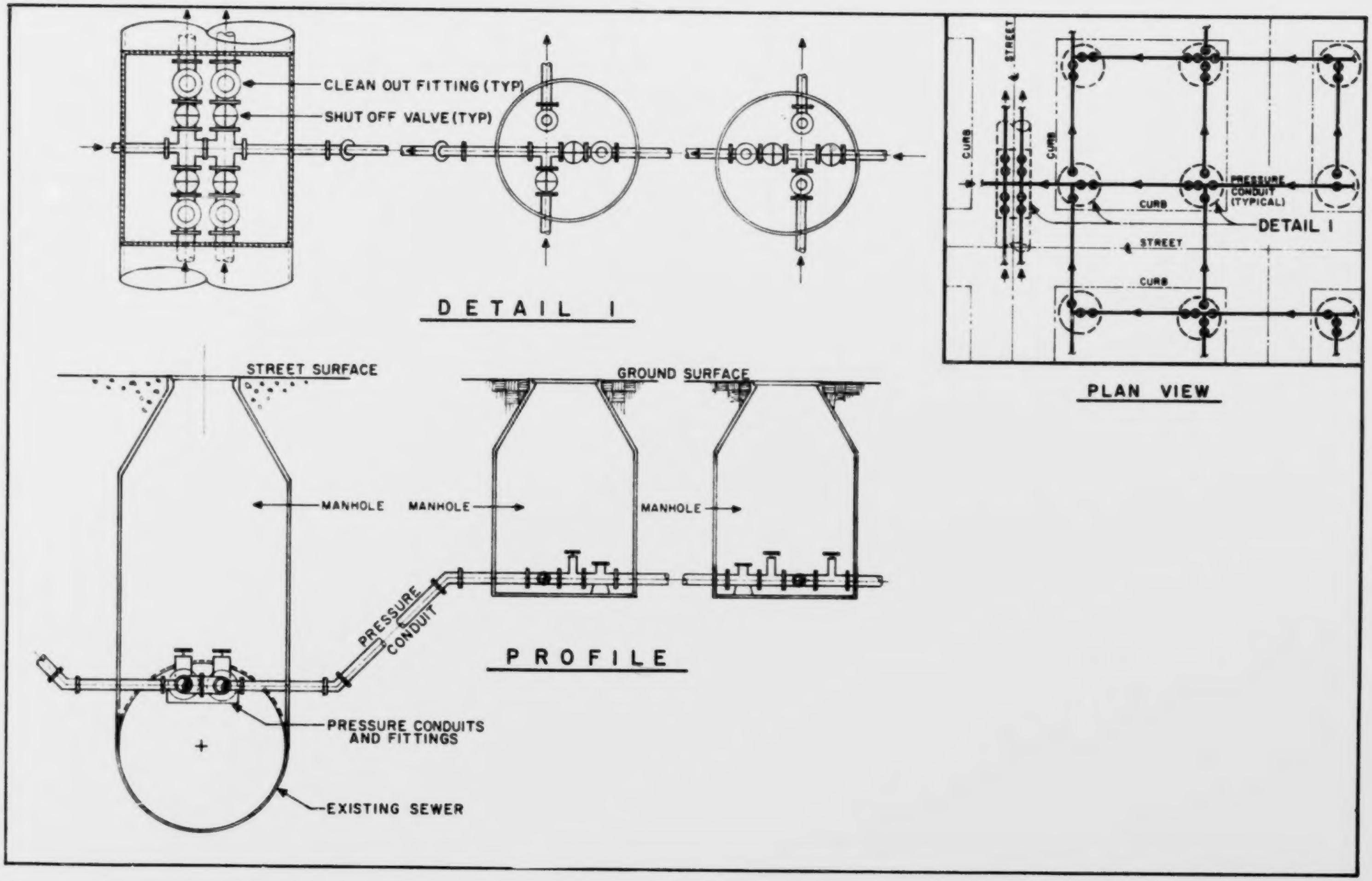
Another factor that suggests the use of zonal grinder-pump stations is the existence of separated sanitary systems which discharge into combined sewers at the terminus of their particular development. It is only logical to maintain the segregated sanitary and storm waters in the upper reaches by constructing a grinder-pump station at the terminus of the existing separated sanitary sewers and pumping the effluent at this point into the new pressurized system.

Several force main layouts were suggested in the ASCE program, varying in costs and degree of flexibility. In general, the systems may be classified as dual or single piping configurations. Only the dual-line layout was considered in this investigation. The dual system consists of a pressure line on each side of the street to which the individual household or building unit may be connected. Each single pipe is sized to convey the peak flow from one side of the street only, representing one half of the total flow that would be generated on the street and transported in a single, common gravity sewer. A typical piping layout is shown in Figure 40. Valving and valve manholes are provided at each intersection to allow isolation of one side of the street for cleaning and maintenance. By manipulation of valves, flows may be passed around any isolated section. Should one section be shutdown for cleaning, the line on the other side of the street would transport all of the flow. This temporary increase in head loss should, however, be of little concern since the additional frictional losses within a block reach are minor when compared to the entire system. The merits of scheduling routine cleaning coincident with off-peak usage are obvious.

A corollary to this method is the use of large-size sewers to provide a passage for the pressure lines rather than trenching along the curb lines. Where the large sewers are available, the pressure lines are suspended from the soffit within the gravity sewer. The method of inserting and suspending the force mains within the larger gravity sewers does not appear to be an easy task, however. Relatively short pipe lengths of ten feet appear to be the most practical for ease of insertion and handling, since the manhole structure would require modification to accommodate the valving, and excavation could be opened adjacent to the manhole to allow for insertion of the short lengths. A sewer dolly, constructed to carry and position the pipe, could transport materials from the access manhole a considerable distance before another access manhole would be required. Construction could only be accomplished during minimum flows.

One intriguing possibility for construction of the in-sewer force main would be the use of formed sections of pipe that fit the soffit of the larger gravity sewer, thus providing a mating shape to the sewer cross-section. The pipe shapes would vary widely to accommodate the required flow and to fit the range of envelope gravity sewer pipe sizes. A circular pipe section could be reformed to almost any desired shape and section. Protective coatings inside and outside would be required to protect the steel from the corrosive atmosphere associated with sewage. A formed rubber gasketed joint at 10-ft intervals would be advisable with pipe supports provided by spaced toggle bolts and angles. Transition sections from arch to round would be required at the valve stations.

Several options were available to store, grind, and pump building and dwelling wastes into the community pressure system. Consideration included ability to pump against a pressure head of about 35 psig. Among the schemes that have been used with some degree of success in past experiments were a storage tank feeding a commercial garbage grinder unit with a separate pump to provide the necessary system head, and a storage tank with a combination grinder-pump unit. The first type of unit has the disadvantages of higher first cost and greater spacial requirements. The combination grinder-pump unit offers lower initial first cost, is compact, and is currently under intense development. The Water Management Laboratory of the General Electric Company has put forth considerable effort toward developing a device capable of meeting this need. Environment One Corporation is producing a similar



CLEAN OUT FITTING (TYP)
SHUT OFF VALVE (TYP)
DETAIL I
STREET SURFACE
GROUND SURFACE
MANHOLE
MANHOLE
MANHOLE
PRESSURE CONDUIT
PROFILE
PRESSURE CONDUITS AND FITTINGS
EXISTING SEWER
CURB
STREET
CURB
CURB
PRESSURE CONDUIT (TYPICAL)
DETAIL I
STREET
CURB
PLAN VIEW

Figure 40. ASCE COMBINED SEWER SEPARATION PROJECT TYPICAL PIPING LAYOUT

appliance for a demonstration grant project with the New York State Department of Health. The Railway Sanitation Research Project has also conducted extensive testing with use of commercial grinders in railcar toilets with a degree of success. Baker Filtration also is experimenting with a combination unit capable of both grinding and pumping at a reasonable cost.

As a comparative figure, it was estimated that a separate grinder and pump would cost approximately twice that of an integrated unit. For this reason, coupled with the present efforts being made to commercially develop and market integrated units, the combination grinder-pump was selected for application to this study.

The household unit power requirements would be in the neighborhood of 1.5 hp while the larger capacity units suitable for apartment buildings would require a 5-hp motor. A 30- to 50-gal storage reservoir is adequate for domestic units and piping is simple. Capacities and heads can be readily varied. Estimated cost of the domestic unit falls in the range of $300 to $500 for the grinder-pump alone, increasing to over $2,000 for a complete operating installation. Figure 41 shows a layout of a typical home installation. The pump discharge line of polyethylene or polybutylene should be equipped with a shutoff valve and check valve. The check may be a normally closed solenoid valve that opens when the grinder-pump starter is energized. This type of check is virtually fool-proof and is essential to avoid backflow and flooding of individual units. The polyethylene or polybutylene discharge tubing has proven successful for snaking through existing house laterals by either pulling or pushing. The fittings for the line should be of the "no-hub" variety to avoid projections within the line.

The concept of zonal pump stations to reduce the number of individual household units results in grinding and pumping installations of a size where a wide range of equipment is available. Comminution equipment with grinding capacities ranging from 0.03 to 20 mgd have been proven in operation for many years with most satisfactory results. The range of sizes for nonclog pumps is almost infinite.

The type of station chosen for the zonal pump stations is the common wet pit-dry pit installation with the grinder, bypass channel and rack, and wet well on one side of the bulkhead. Vertical pedestal mounted pumps and electrical switchgear are located in the dry pit side of the station. Consideration may be given to installation of flow meters at each station. A magnetic flow meter installed in the discharge piping would be suitable for maintaining flow records. The pump station configuration is purposely long and narrow to allow installation between curb and property line in many instances. A typical installation of this type is depicted in Figure 42. The larger pump stations, as shown in Figures 43 and 44, where two or more grinders and a bypass channel are required, precluded the use of a narrow structure.

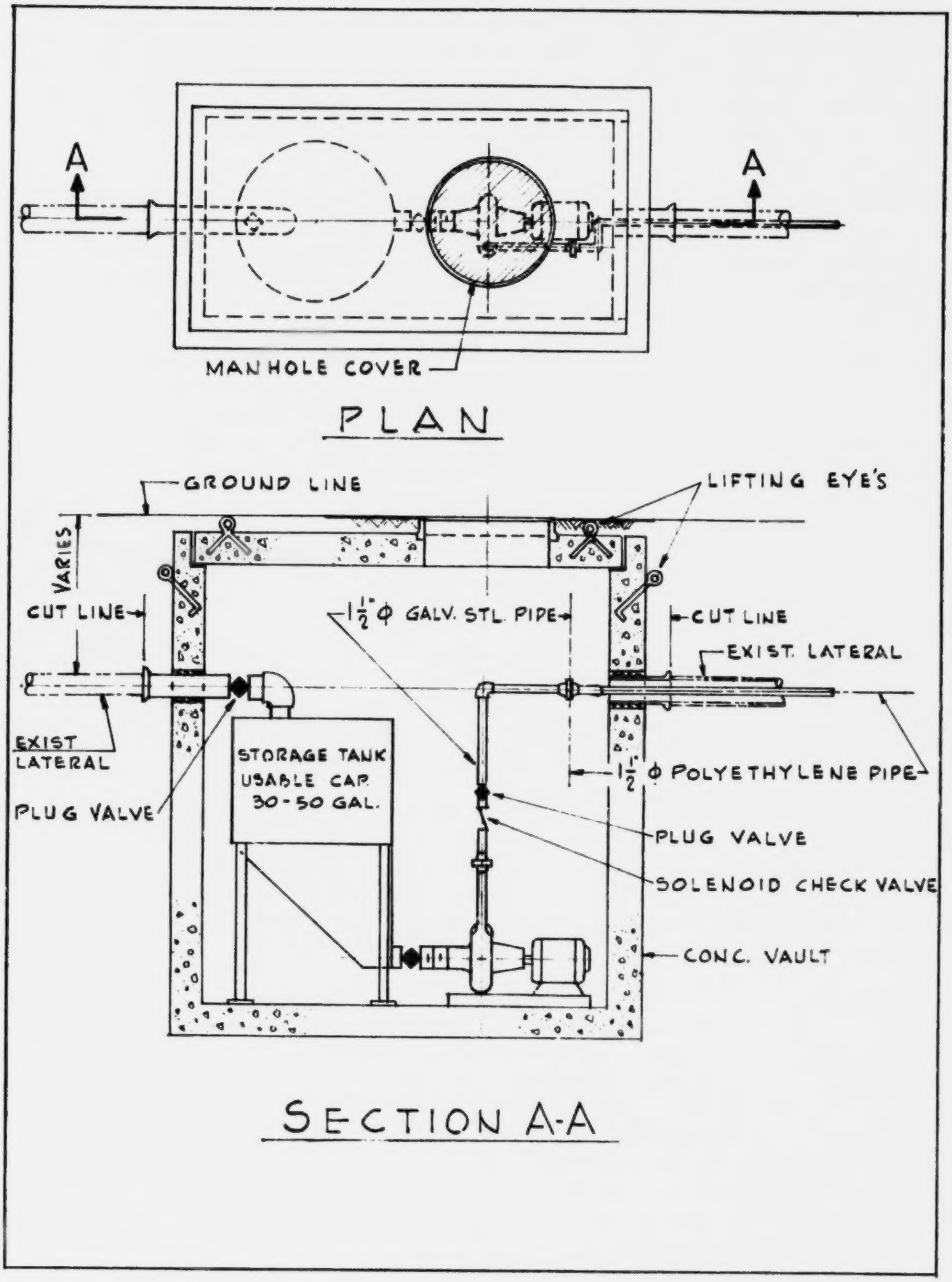
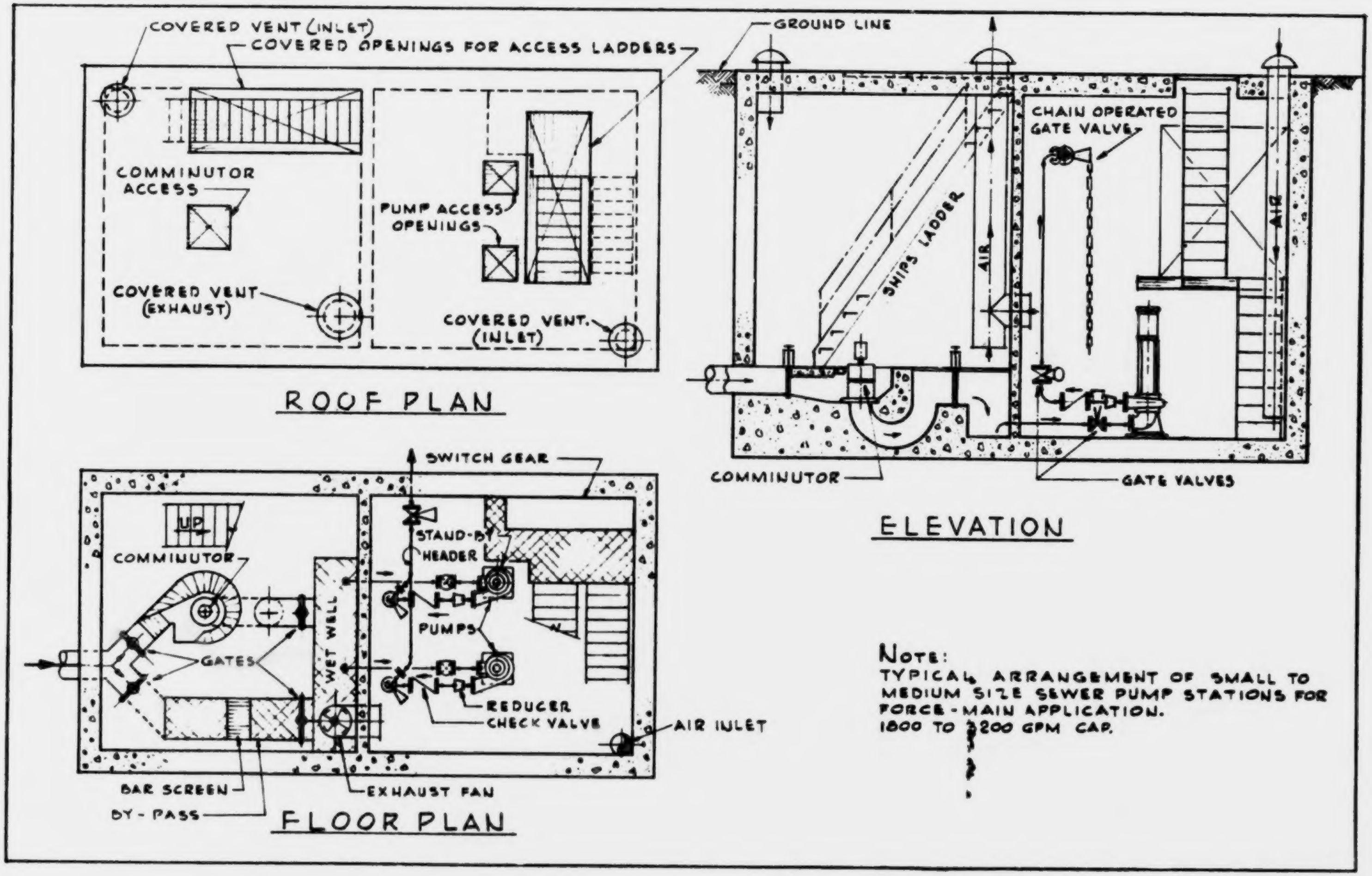
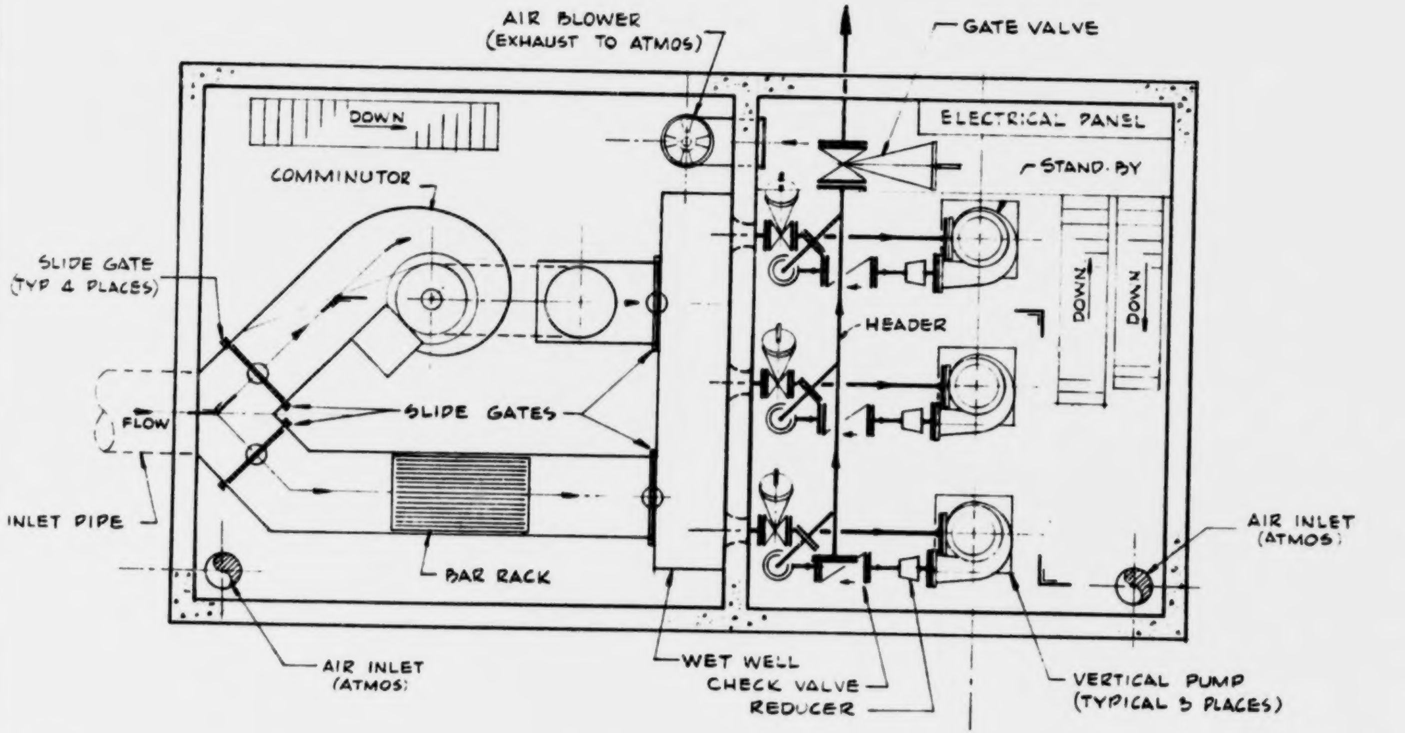

A
A
MANHOLE COVER
PLAN
GROUND LINE
LIFTING EYE'S
VARIES
CUT LINE
$1\frac{1}{2}$" Φ GALV. STL. PIPE
CUT LINE
EXIST. LATERAL
EXIST LATERAL
STORAGE TANK
USABLE CAP.
30-50 GAL.
$1\frac{1}{2}$ Φ POLYETHYLENE PIPE
PLUG VALVE
PLUG VALVE
SOLENOID CHECK VALVE
CONC. VAULT
SECTION A-A

Figure 41. INDIVIDUAL GRINDER-PUMP UNIT


COVERED VENT (INLET)
COVERED OPENINGS FOR ACCESS LADDERS
COMMINUTOR ACCESS
PUMP ACCESS OPENINGS
COVERED VENT (EXHAUST)
COVERED VENT. (INLET)
ROOF PLAN
GROUND LINE
CHAIN OPERATED GATE VALVE
SHIPS LADDER
AIR
AIR
COMMINUTOR
GATE VALVES
ELEVATION
SWITCH GEAR
UP
COMMINUTOR
STAND-BY HEADER
PUMPS
GATES
WET WELL
REDUCER
CHECK VALVE
AIR INLET
BAR SCREEN
BY-PASS
EXHAUST FAN
FLOOR PLAN

NOTE:
TYPICAL ARRANGEMENT OF SMALL TO MEDIUM SIZE SEWER PUMP STATIONS FOR FORCE-MAIN APPLICATION.
1800 TO 3200 GPM CAP.

Figure 42. MEDIUM ZONAL GRINDER-PUMP STATION


AIR BLOWER (EXHAUST TO ATMOS)
GATE VALVE
ELECTRICAL PANEL
STAND-BY
DOWN
DOWN
DOWN
COMMINUTOR
SLIDE GATE (TYP 4 PLACES)
HEADER
SLIDE GATES
FLOW
INLET PIPE
BAR RACK
AIR INLET (ATMOS)
AIR INLET (ATMOS)
WET WELL
CHECK VALVE
REDUCER
VERTICAL PUMP (TYPICAL 3 PLACES)

NOTE

TYPICAL ARRANGEMENT OF LARGE SIZE SEWER PUMP STATIONS FOR FORCE-MAIN APPLICATION. 12,000 GPM CAP.

Figure 43. PLAN OF LARGE GRINDER-PUMP STATION

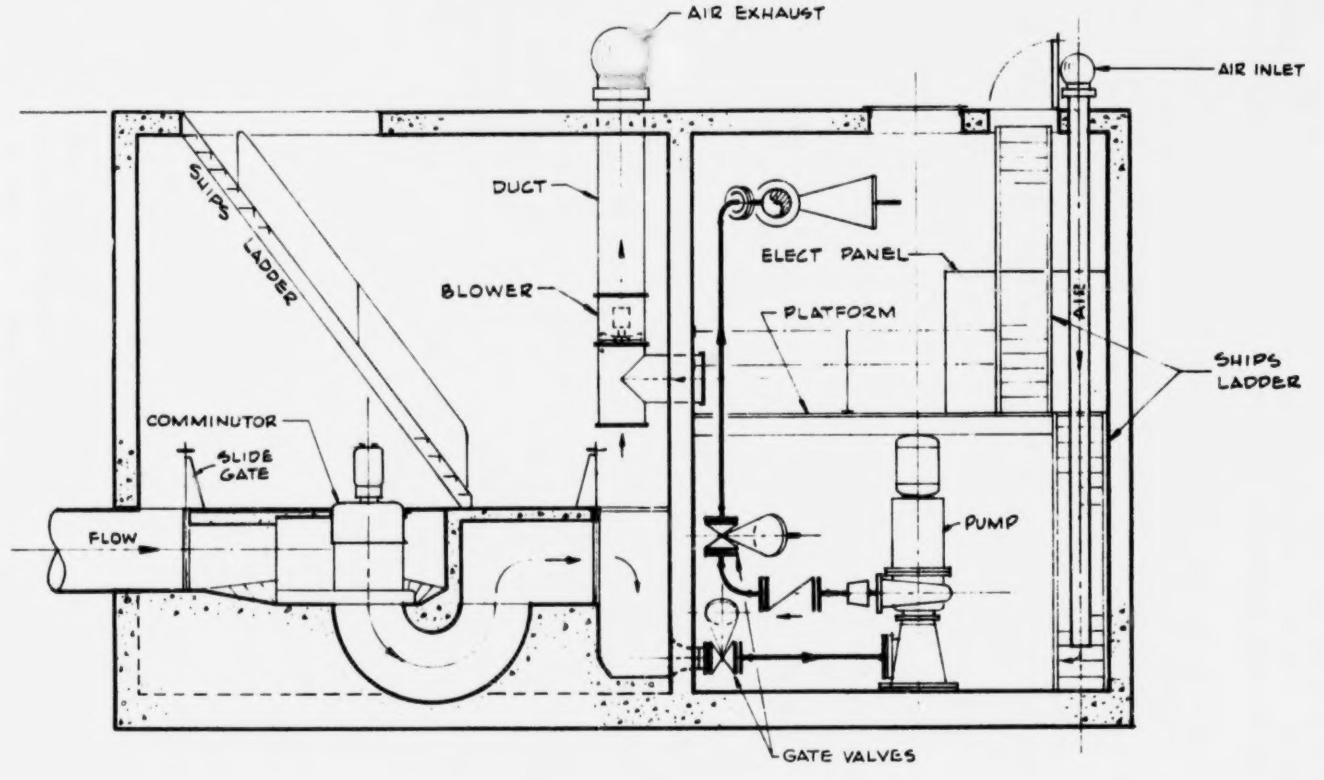

AIR EXHAUST
AIR INLET
DUCT
ELECT PANEL
BLOWER
PLATFORM
SHIPS LADDER
AIR
SHIPS LADDER
COMMINUTOR
SLIDE GATE
FLOW
PUMP
GATE VALVES

Figure 44. ELEVATION OF LARGE GRINDER-PUMP STATION

To avoid a large wet well and the usual odor nuisance, variable speed pumps were used in all stations. The variable speed equipment matches the incoming flow and eliminates the surges associated with intermittent pump operation. All pump stations were considered to be below surface installations with separate hatches for the wet well and dry well.

Four alternatives were investigated relative to the ASCE force main concept. As with the first three gravity conveyance alternatives, the force main alternatives are primarily concerned with the Combined Area portion of the Study Area. Thus, no attempt was made to adapt the force main concept to the parts of the Study Area where separation of flows already exists. The so-called hybrid force main concepts utilize a combination of gravity flow and pressurized flow with zonal-type grinder-pump units rather than units located at every individual service connection.

Total Force Main

For the Combined Area, a total force main concept was applied, wherein every service connection without exception within the area would be served with a grinder-pump unit so that sanitary flows would be handled entirely via pressure lines. In the Separated Area, the sanitary sewage would flow in the existing gravity sanitary sewers to the pressure system boundary where grinder-pump units would be located to inject it into the pressurized system.

Hybrid Force Main (New Gravity Storm Sewers Area)

For the new gravity storm sewer system previously described, it was assumed that storm runoff would travel on the surface for about one block before catch basins or inlets would intercept the flow and discharge to the sewer. In superimposing that alternative system on the existing combined network, there remains many laterals of one-block length not required for storm flows and, therefore, available for sanitary use. But in applying the Total Force Main concept, these existing laterals would be unused. A logical force main scheme would entail making use of these existing laterals for sanitary gravity flow to zonal grinder-pump units. This Hybrid Force Main alternative embodies this approach wherein the new sanitary force main would be installed only where the new gravity storm sewers would be placed. All force mains in this system would be placed in trenches.

Hybrid Force Main (Pipe-in-Pipe/New Gravity Sanitary Sewers Upstream)

A basic feature of the ASCE Force Main concept is the utilization of existing combined sewers as a conduit or pipe gallery for the installation of the new smaller sanitary pressure lines so that storm and sanitary flows become separated. Based on the design criteria adopted for this study, the existing combined system was found to be undersized for storm flows along almost its entire length. As a result, the pipe-in-pipe concept has little advantage, since major storm water conveyance system augmentation was indicated to provide adequate hydraulic capacity and no excess capacity within the pipe is available for placement of even a small pressure conduit.

This alternative considered use of the force main only within a 48-in. diameter or larger gravity conduit. Since the existing combined system was already undersized, this force main system alternative was superimposed on the new gravity storm sewer system downstream of the locations specifying a 48-in. or larger diameter pipe for the conveyance of separated storm water runoff.

This hybrid system requires the installation of a new gravity sewer system for separation of flows upstream from the first downstream 48-in. pipe. For this alternative, a new upstream gravity sanitary sewer system was selected and the existing sewer was utilized and augmented for separated storm flows.

Hybrid Force Main (Pipe-in-Pipe/New Gravity Storm Sewers Upstream)

This alternative is identical to that immediately preceding with the exception that a new gravity storm system was installed upstream with the existing upstream combined sewer utilized and augmented where necessary for separated sanitary flows.

COSTS

Table 17 summarizes the costs for each of the seven conveyance alternatives for the Study Area, including the augmentation requirements for storm water runoff conveyance in the Separated Area. All seven alternative subsystems provide the same hydraulic capacity by a combination of new facilities, existing facilities, and augmentations of existing facilities. The capital costs have been converted to an annual basis and annual operational and maintenance costs have been included. Since all seven alternatives involve new or augmented facilities for handling storm flows, the costs were computed for 2-, 5-, and 10-year storm recurrence expectancies. For the four force main concepts, the costs include the grinder-pump requirements associated with each of the alternatives.

COMPARISON

All seven conveyance alternatives were designed to meet the same specified criteria, including augmentation of the existing sewer system where found necessary to comply. Therefore, all seven alternatives may be said to provide equal performance characteristics.

Alternative No. 1 provides a new-design storm sewer system and Alternative No. 2 provides a new-design sanitary sewer system, with the same complete separation of flows occurring in both cases. Alternative No. 2 was eliminated from further consideration, since it is more costly than Alternative No. 1. Also, complete separation via the installation of a new-design storm sewer system follows the pattern already well established within the Study Area for relief of the existing combined sewer system. Approximately one-third of the Study Area is served by separated storm sewers with the older combined sewers being utilized for sanitary sewage.

Table 17

ANNUAL COSTS OF CONVEYANCE ALTERNATIVES
(millions of dollars)

General System Description	Storm Recurrence Expectancy		
	2-yr	5-yr	10-yr
1. New Gravity Storm Sewers (Augmented Existing System for Sanitary)	2.68	3.68	4.16
2. New Gravity Sanitary Sewers (Augmented Existing System for Storm)	3.35	4.12	4.48
3. Augmentation of Existing Combined Sewers	2.16	3.10	3.48
4. Total Force Main	18.74	19.52	19.87
5. Hybrid Force Main (New Gravity Storm Sewers Area)	13.78	14.56	14.91
6. Hybrid Force Main (Pipe-in-Pipe/New Gravity Sanitary Sewers Upstream)	5.33	6.51	6.94
7. Hybrid Force Main (Pipe-in-Pipe/New Gravity Storm Sewers Upstream	4.70	5.92	6.35

Alternative No. 3 proposes continuing the combination of sanitary and storm flows with the existing system augmented to meet the specified criteria. Although less costly than Alternative No. 1, a direct comparison between the two systems is not possible since subsequent treatment requirements are not identical. Therefore, both alternatives were considered within the framework of overall sewerage systems evaluation.

Alternatives No. 4 through 7 require a new-design sanitary force main system resulting in complete separation of flows in all four cases.

Alternatives No. 4 and 5 propose large-scale force main installations over virtually the entire existing truly combined portion of the Study Area. Of the two systems, Alternative No. 5 is more economical, primarily because of more efficient utilization of the existing combined sewer system. Therefore, Alternative No. 4 was eliminated from further consideration.

Alternatives No. 6 and 7 propose new force mains only where a storm sewer of 48-in. diameter or larger is available for placement of the pressure lines within. Using a new-design storm sewer system as the basis, a relatively small portion of the combined sewer system area meets the 48-in. diameter requirement. As a result, these two alternatives are far less costly than the large-scale force main systems and they approach the still lower cost gravity systems. Of the two, Alternative No. 7 is less costly; therefore, Alternative No. 6 was eliminated from further consideration.

In defense of the force main concept, a more detailed investigation than was performed in this study would probably show numerous ways to reduce system costs. For example, elimination of dual lines and elimination of the grinder requirement where flows exceed an established minimum would provide significant savings over those developed in this study. It is estimated, for example, that about a 20-percent reduction in annual cost could result if a single-line force main system were used. However, except where unusual conditions prevail, it appears doubtful that a force main system could be justified on the basis of cost alone in competition with a more conventional gravity sewer system.

The conveyance system cost associated with any overall sewer system that is predicated on separation of flows would be independent of the remainder of the total system costs. On this basis, all force main systems could be eliminated, since separated gravity flow systems are less expensive. However, Alternatives No. 5 and 7 were maintained in the analysis and considered as representative force main concepts in the development of overall sewage system costs.

STORAGE

Three of the candidate systems, including stabilization ponds which are primarily a treatment process, are considered here under the storage category. Since a complete sewer system does not necessarily require storage, this system component may be considered as optional. Thus four alternatives were investigated, of which one of the four was the complete absence of storage facilities.

Regardless of the storage alternative under consideration for containment of combined sewage, the reservoirs were sized to accommodate only the combined flow volume occurring during periods of storm runoff. At all other times, the sanitary flow would bypass the reservoir and would be processed through the conventional treatment facilities.

DESCRIPTION

A storage facility provides a means for modulating peak flows, thus reducing downstream capacity requirements for conveyance, treatment, and disposal and providing for more efficient utilization of downstream equipment and facilities. These obvious advantages must be weighed against the cost of the reservoir installation.

Suitable locations for reservoirs are frequently difficult to find. Usually the area involved is already developed, resulting in high land costs and strong objections from vested interests to the construction of this type of facility. Thus, regardless of potential advantages, it may become necessary to provide sewer system improvements without benefit of storage.

Underground Storage

This concept evolved from the development work carried out in Chicago on underground storage of combined sewage. The Chicago program proposed to utilize modern tunnel excavation techniques to provide new main sewers and interceptors as well as mined storage. The stored waters could then be released at a reduced rate to treatment facilities. The costs of the long-range program are to be defrayed in part by power generation through pumped storage between underground and surface reservoirs and by sale of mined aggregate.

The subsurface conditions in the Sacramento area do not favor the deep tunnel approach. Consequently, the concept was modified to consider only relatively shallow or near-surface underground storage provided through open-type excavation rather than by tunneling.

Underground reservoirs are best justified in already developed areas. A location just north of Sump No. 2 was selected for application of this concept (see Figure 25). This prospective site is presently single-family residential. Extensive relocation would be required, with the resulting open area over the reservoir available for development of recreational areas, public parks, or other applications.

A reinforced concrete structure is proposed with the top of sufficient strength to withstand earth cover and light-weight vehicular loads. A dividing wall would provide two separated water-tight compartments to assist in operation and maintenance activities. Access provisions for periodic cleanout would be provided.

Surface Storage

Assuming the availability of flat open land, a surface reservoir provides the least costly storage facility. Cut-and-fill construction is envisioned with the fill material utilized for peripheral dike construction. A dike dividing the reservoir into two parts is proposed to assist in operation and maintenance activities. An asphaltic pavement surfacing would be used to restrict outflow from and inflow to the reservoir due to groundwater seepage.

For the Sacramento area, three locations were selected for application of the surface reservoir concept. The American River Park location would provide storage for already separated storm flows only. Either the Sacramento Port location or the South Sacramento location would provide storage for several combinations of storm flow alone and combined flow from either the Combined Area or the total Study Area.

Stabilization Pond

The preceding description of surface storage facilities applies equally to stabilization ponds. Considered primarily as a treatment process, the stabilization ponds were accordingly sized to a large capacity. The storage reservoirs, on the other hand, primarily would serve as peak flow modulators where capacity is balanced against an optimum outflow rate.

Both the Sacramento Port and the South Sacramento locations were used for application of the stabilization pond concept. The combined flow pond was sized on the basis of a rough estimate of available acreage. The capacity of the stabilization pond for treatment of separated storm water runoff was determined on the basis of the same pond withdrawal rate that was used for combined flow.

COSTS

Costs were developed for a range of reservoir sizes and depths which, in turn, were related to a given reservoir outflow or withdrawal rate. Table 18 presents reservoir capacities for the selected locations and

Table 18

CAPACITIES OF RESERVOIRS
(millions of gallons)

Location	Withdrawal Rate, mgd	2-yr Storm Recurrence				5-yr Storm Recurrence				10-yr Storm Recurrence			
		Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow
American River Park		Separated Area				Separated Area				Separated Area			
Surface Reservoir	10	176				264				325			
	20	117				176				222			
	40	73				108				130			
	80	39				55				64			
	160	12				21				30			
	231*	0				-				-			
	320	0				0				1			
	352*	-				0				-			
	422*	-				-				0			
Sump 2 Vicinity		Combined Area		Study Area		Combined Area		Study Area		Combined Area		Study Area	
Underground Reservoir	40	305	619	528	1108	457	863	766	1564	588	1010	912	1857
	80	190	352	365	505	306	489	547	717	392	570	684	896
	160	104	153	205	290	156	215	293	430	193	257	342	554
	320	42	57	96	121	68	90	147	150	91	117	183	218
	575*	0	-	-	-	-	-	-	-	-	-	-	-
	577*	0	-	-	-	-	-	-	-	-	-	-	-
	640	0	0	20	28	9	15	47	62	13	20	68	93
	842*	-	-	-	-	-	0	-	-	-	-	-	-
	892*	-	-	-	-	0	-	-	-	-	-	-	-
	993*	-	-	-	-	-	-	-	-	-	0	-	-
	1077*	-	-	-	-	-	-	-	-	0	-	-	-
Sacramento Port		Combined Area		Study Area		Combined Area		Study Area		Combined Area		Study Area	
Surface Reservoir	10	847	1857	-	-	1205	2737	-	-	1401	3128	-	-
	40	305	619	528	1108	457	863	766	1564	588	1010	912	1857
	80	190	352	365	505	306	489	547	717	392	570	684	896
	160	104	153	205	290	156	215	293	430	193	257	342	554
	320	42	57	96	121	68	90	147	150	91	117	183	218
	640	0	0	20	28	9	15	47	62	13	20	68	93
South Sacramento		Combined Area		Study Area		Combined Area		Study Area		Combined Area		Study Area	
Surface Reservoir	10	847	1857	-	-	1205	2737	-	-	1401	3128	-	-
	40	305	619	528	1108	457	863	766	1564	588	1010	912	1857
	80	190	352	365	505	306	489	547	717	392	570	684	896
	160	104	153	205	290	156	215	293	430	193	257	342	554
	320	42	57	96	121	68	90	147	150	91	117	183	218
	640	0	0	20	28	9	15	47	62	13	20	68	93

*Peak Flow (No Reservoir)

2-, 5-, and 10-year storm recurrence expectancies. Corresponding annual costs, including operation and maintenance, are summarized in Table 19.

To simplify the assembly of major subsystem components into complete sewer systems, the reservoir costs also include the associated pumping and connecting piping costs. Underground reservoirs were assumed to have gravity inflow and pumped withdrawal. A typical pumping station for reservoir withdrawal is shown in Figures 45 and 46. Surface reservoirs were assumed to have pumped inflow and gravity outflow. Depicted in Figures 47, 48, and 49 is a typical below-grade pumping station which may be utilized in a variety of applications including the filling of reservoirs. Connecting piping is from the termination of the conveyance systems at Sump No. 2 to the reservoir. Where already separated storm flows from the American River Park location are transported to Sump No. 2 and beyond, the cost of this piping was also borne by the associated reservoir. The special case at Sump No. 2 where outflow rate equals incoming peak flow, requiring a zero reservoir capacity, was also included to account for these same pumping and piping requirements under the alternative or no reservoir. Utilization of either the Sacramento Port or the South Sacramento site with no reservoir was not considered.

COMPARISON

From the standpoint of providing storage, all of the reservoir alternatives may be considered as giving equal performance characteristics for the given criteria. The optimum reservoir size for any specific situation with regard to either subsequent treatment process or to physical location is related to the reservoir outflow rate. This is discussed subsequently under Trade-Off Optimization. While not explored in this study, the problem of objectionable anaerobic conditions developing where stored flows are impounded for an extended period must also be considered in relation to reservoir size and outflow rate.

Underground reservoirs are far more costly to construct than surface reservoirs. This higher cost must be balanced against the advantages of the underground system in comparing the two alternatives. An obvious advantage of the underground installation is that it may be visually hidden so that objections stemming from appearance are reduced. Objectionable odors are also better controlled within an enclosed structure.

TREATMENT PROCESS

Two of the candidate systems, dissolved air flotation and mechanical screening, are considered here. For convenience, wastewater constituent reductions effected by reservoirs or stabilization ponds and chlorination have not been considered under this heading.

Table 19

ANNUAL COSTS OF RESERVOIRS
(millions of dollars)

Location	With-drawal Rate, mgd	2-yr Storm Recurrence				5-yr Storm Recurrence				10-yr Storm Recurrence			
		Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow
American River Park		Separated Area				Separated Area				Separated Area			
Surface Reservoir	10	1.09				1.64				1.96			
	20	1.03				1.55				1.84			
	40	.93				1.42				1.68			
	80	.87				1.29				1.55			
	160	.80				1.23				1.47			
	231*	.77				-				-			
	320	-				1.17				1.42			
	352*	-				1.16				-			
	422*	-				-				1.39			
Sump 2 Vicinity		Combined Area		Study Area		Combined Area		Study Area		Combined Area		Study Area	
Underground Reservoir	40	2.82	4.87	5.71	9.41	3.98	6.65	7.52	12.74	4.98	9.20	8.73	14.87
	80	1.93	3.12	4.26	5.35	2.77	4.10	6.08	7.19	3.56	4.63	7.23	8.77
	160	1.25	1.63	3.11	3.81	1.72	2.12	4.28	5.31	2.02	2.47	4.96	6.51
	320	.80	.93	2.36	2.56	1.06	1.23	3.25	3.28	1.29	1.48	3.85	4.11
	575*	.39	-	-	-	-	-	-	-	-	-	-	-
	640	-	.54	-	-	.67	.73	2.60	2.74	.73	.82	3.09	3.31
	577*	-	.39	-	-	-	-	-	-	-	-	-	-
	842*	-	-	-	-	-	.54	-	-	-	-	-	-
	892*	-	-	-	-	.57	-	-	-	-	-	-	-
	993*	-	-	-	-	-	-	-	-	-	.62	-	-
	1077*	-	-	-	-	-	-	-	-	.68	-	-	-
Sacramento Port		Combined Area		Study Area		Combined Area		Study Area		Combined Area		Study Area	
Surface Reservoir	10	1.41	2.35	-	-	2.03	3.34	-	-	2.35	3.89	-	-
	40	1.18	1.73	2.80	3.95	1.76	2.47	3.96	5.43	2.09	2.87	4.64	6.41
	80	.93	1.25	2.50	2.80	1.46	1.79	3.54	3.88	1.76	2.01	4.22	4.64
	160	.77	.87	2.19	2.38	1.15	1.24	3.05	3.39	1.34	1.41	3.55	4.06
	320	.65	.68	1.95	2.07	.99	1.00	2.80	2.94	1.16	1.15	3.25	3.47
	640	.55	.55	1.82	1.88	.87	.85	2.61	2.65	1.01	.96	3.03	3.15
South Sacramento		Combined Area		Study Area		Combined Area		Study Area		Combined Area		Study Area	
Surface Reservoir	10	2.33	3.27	-	-	3.31	4.62	-	-	3.82	5.28	-	-
	40	2.10	2.66	3.82	5.18	3.04	3.75	5.42	7.05	3.56	1.25	6.38	8.57
	80	1.86	2.17	3.51	4.03	2.70	3.07	5.01	5.50	3.23	3.40	5.95	6.83
	160	1.72	1.79	3.21	3.61	2.43	2.53	4.52	5.01	2.82	2.79	5.29	6.23
	320	1.57	1.60	2.97	3.30	2.27	2.28	4.26	4.46	2.63	2.53	4.98	5.64
	640	1.48	1.48	2.83	3.11	2.15	2.13	4.07	4.28	2.48	2.34	4.77	5.31

*Peak Flow (No Reservoir)

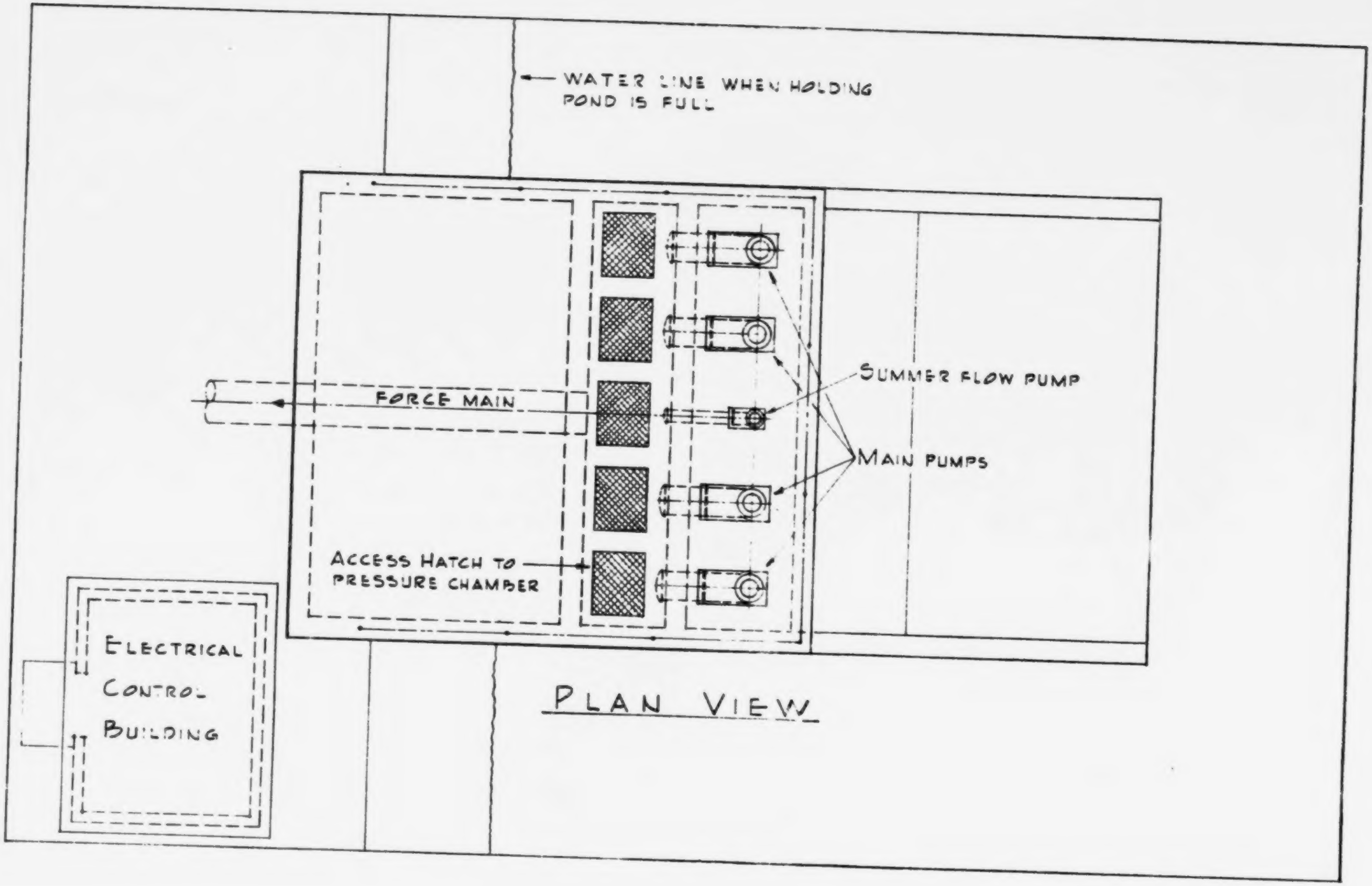

WATER LINE WHEN HOLDING POND IS FULL
SUMMER FLOW PUMP
MAIN PUMPS
FORCE MAIN
ACCESS HATCH TO PRESSURE CHAMBER
ELECTRICAL CONTROL BUILDING
PLAN VIEW

Figure 45. PLAN OF TYPICAL PUMP STATION FOR RESERVOIR WITHDRAWAL

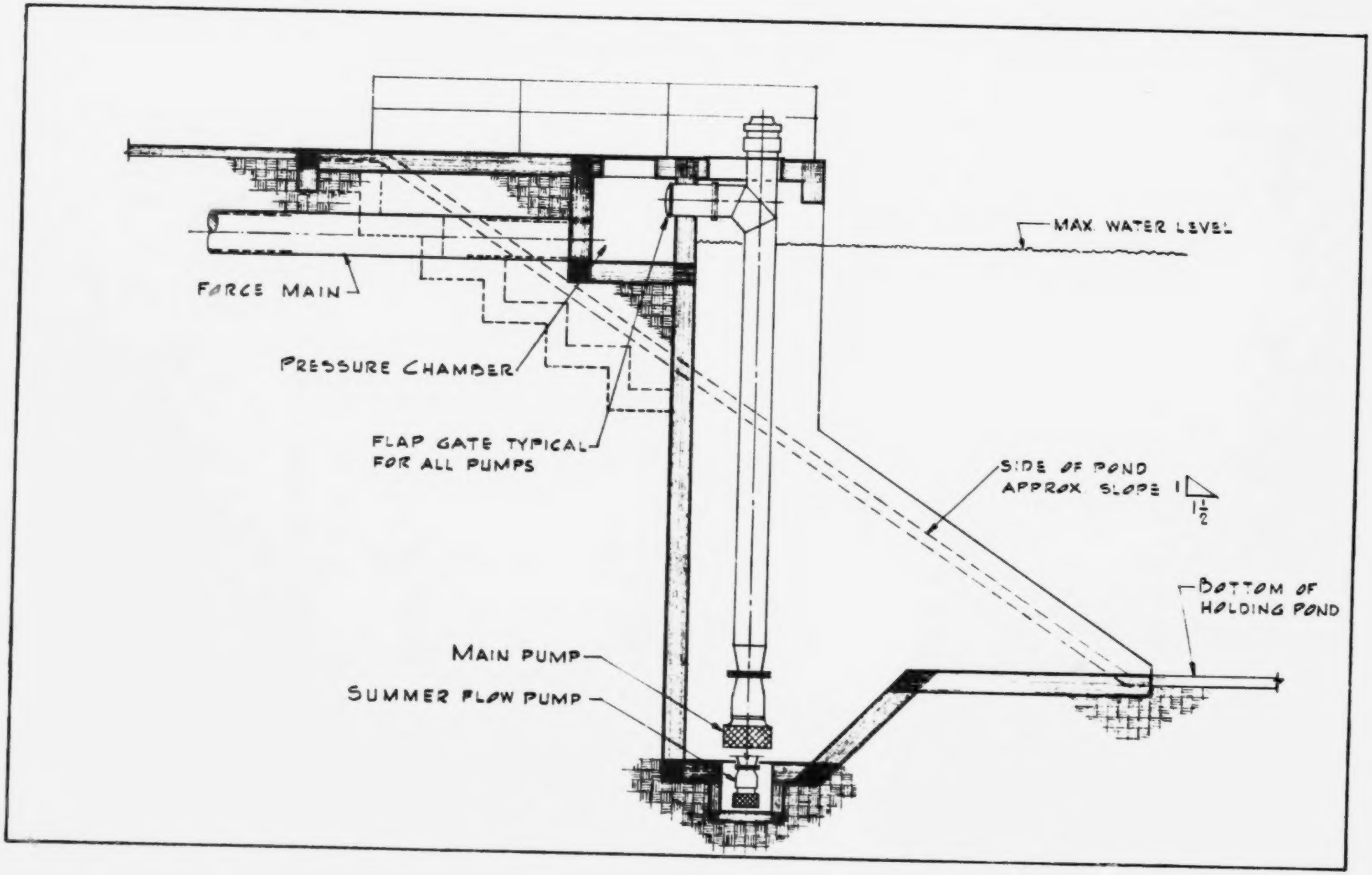

MAX. WATER LEVEL
FORCE MAIN
PRESSURE CHAMBER
FLAP GATE TYPICAL FOR ALL PUMPS
SIDE OF POND APPROX. SLOPE 1 / 1 1/2
BOTTOM OF HOLDING POND
MAIN PUMP
SUMMER FLOW PUMP

Figure 46. SECTION OF TYPICAL PUMP STATION FOR RESERVOIR WITHDRAWAL

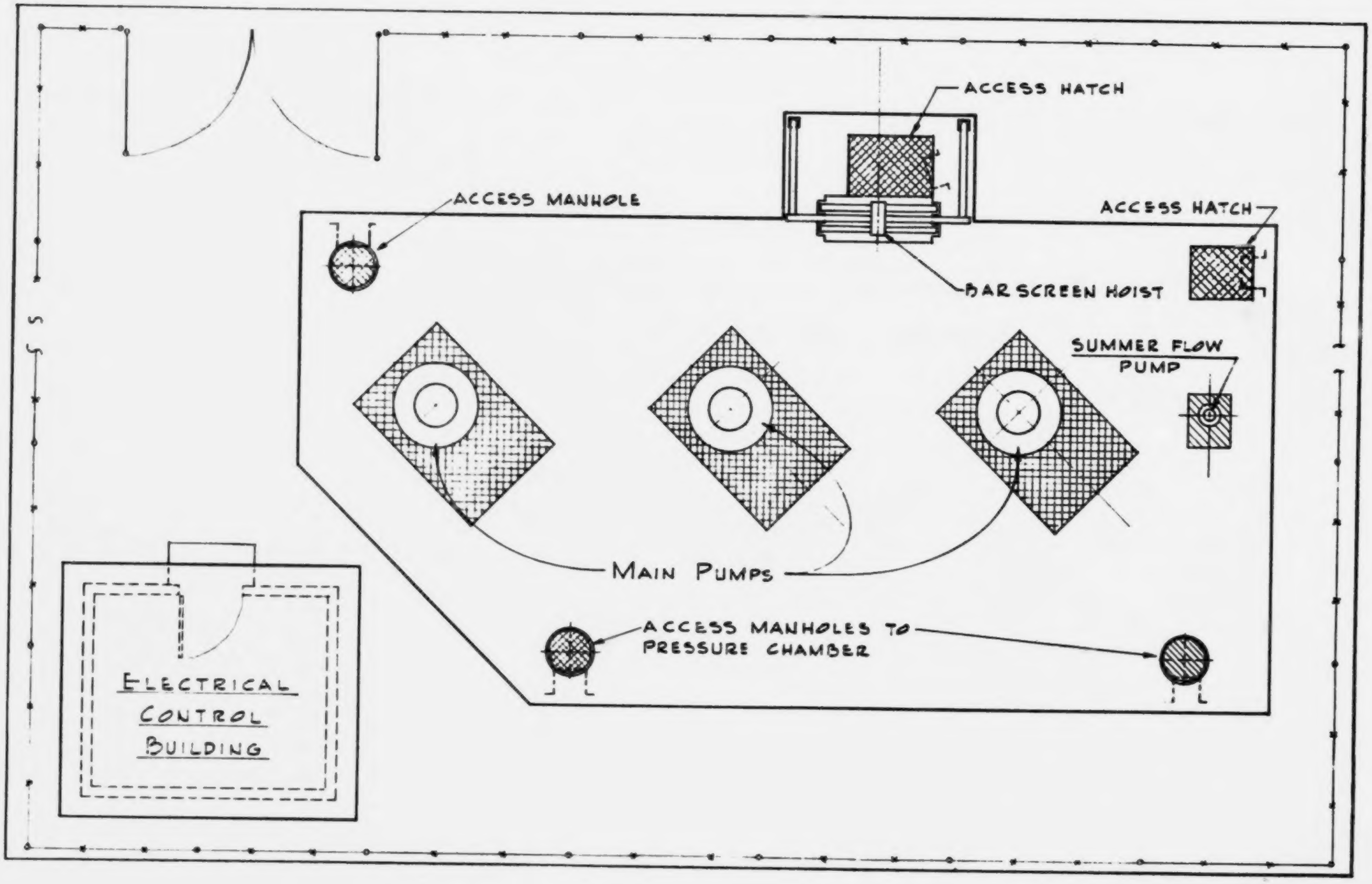

ACCESS HATCH
ACCESS MANHOLE
ACCESS HATCH
BAR SCREEN HOIST
SUMMER FLOW PUMP
MAIN PUMPS
ACCESS MANHOLES TO PRESSURE CHAMBER
ELECTRICAL CONTROL BUILDING

Figure 47. PLAN AT GRADE OF TYPICAL SUBSURFACE PUMP STATION

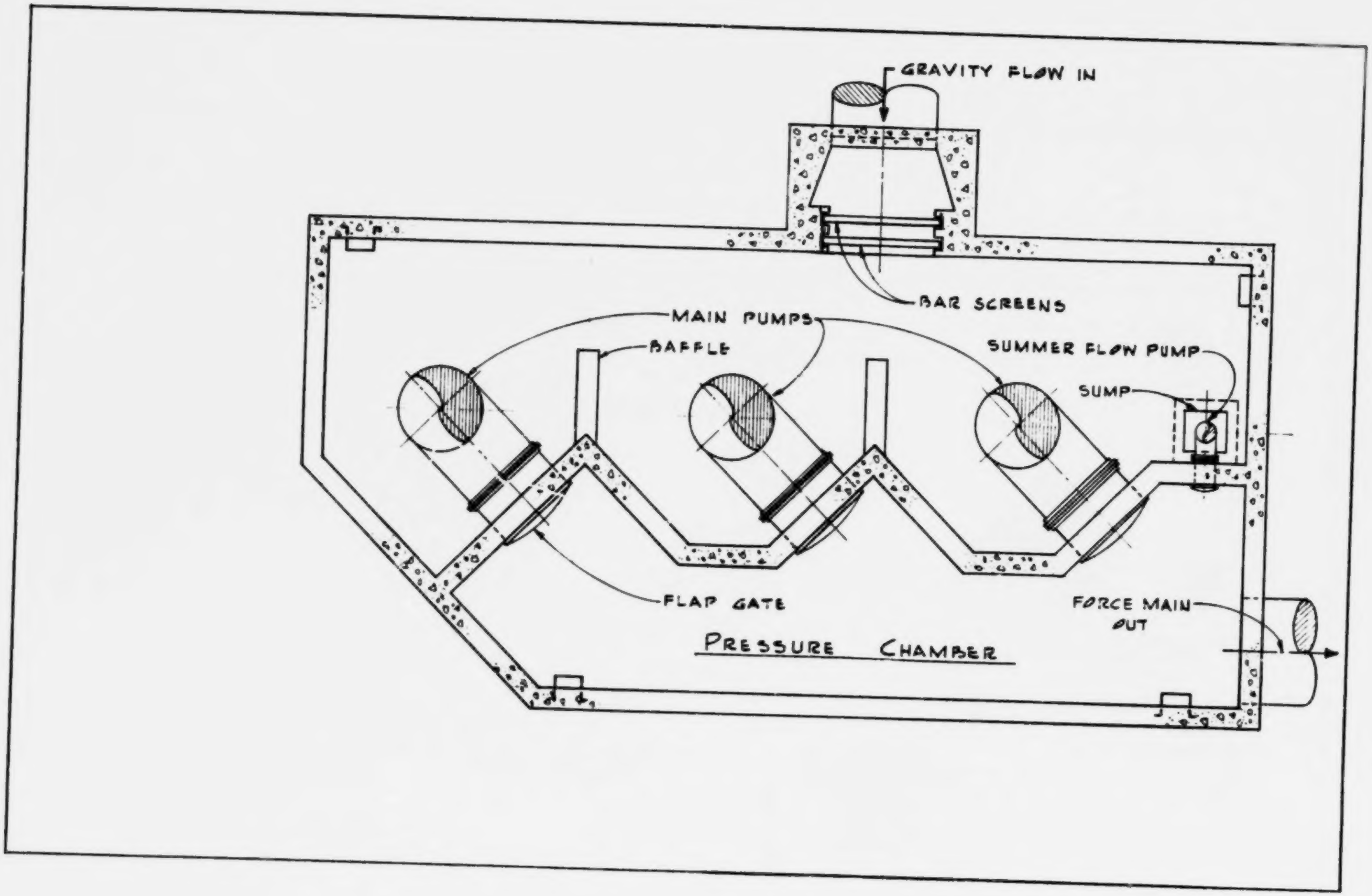

GRAVITY FLOW IN
BAR SCREENS
MAIN PUMPS
BAFFLE
SUMMER FLOW PUMP
SUMP
FLAP GATE
FORCE MAIN OUT
PRESSURE CHAMBER

Figure 48. PLAN BELOW GRADE OF TYPICAL SUBSURFACE PUMP STATION

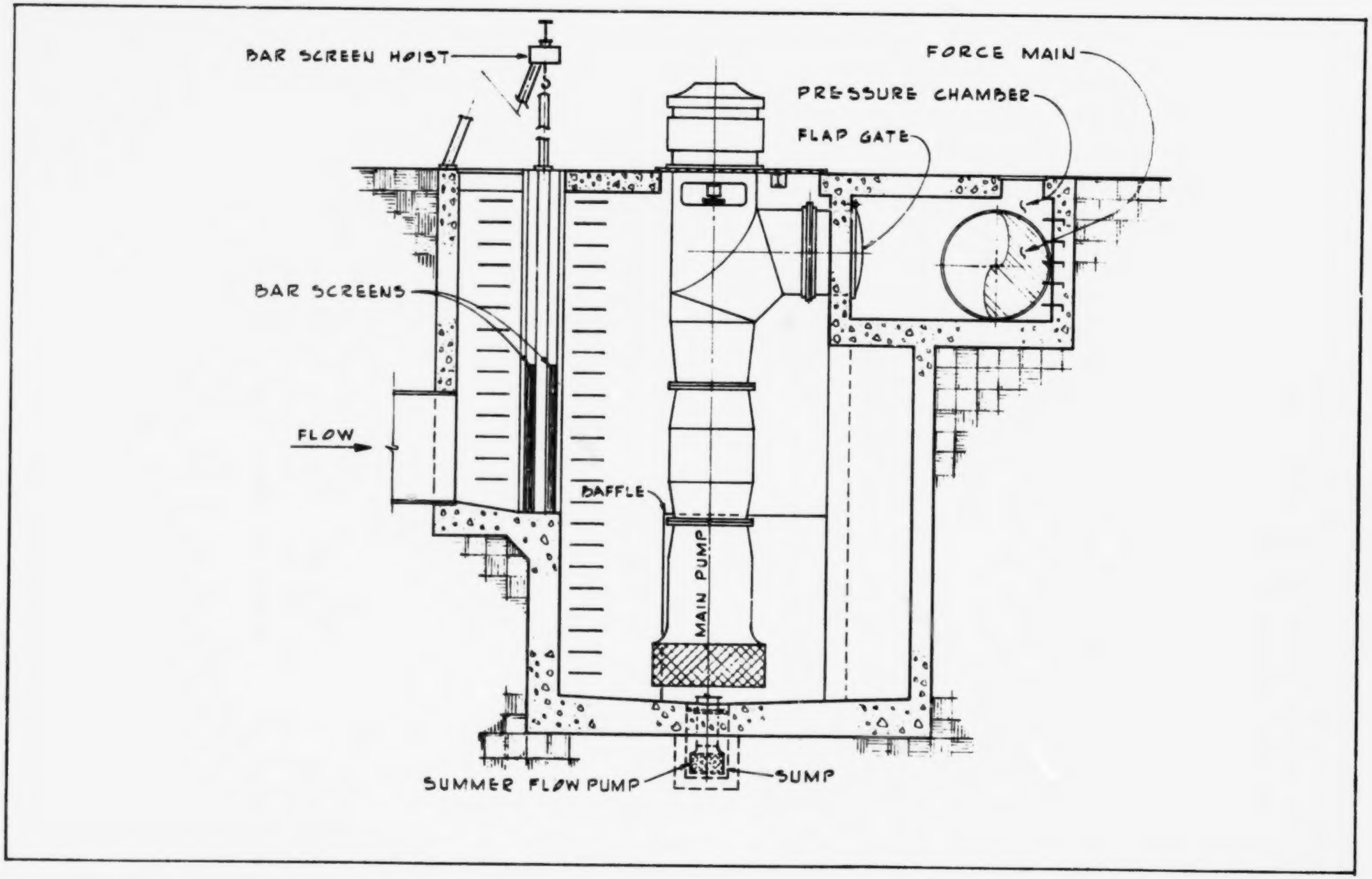

BAR SCREEN HOIST
FORCE MAIN
PRESSURE CHAMBER
FLAP GATE
BAR SCREENS
FLOW
BAFFLE
MAIN PUMP
SUMMER FLOW PUMP
SUMP

Figure 49. SECTION OF TYPICAL SUBSURFACE PUMP STATION

The treatment alternatives consider only the processing of either combined flows or already separated storm flows. In all instances where only sanitary sewage would be present in the sewer, the flow would be diverted to the conventional treatment facilities and is not further considered.

DESCRIPTION

No Treatment

Within the framework of this study, the no-treatment alternative considers only the absence of either dissolved air flotation or mechanical screening as candidate system treatment processes. The option remains of utilizing a pond, either individually or in conjunction with conventional treatment, with the resulting improvement in waste discharge characteristics.

In the cases predicated on separation of flows, it was presumed that all sanitary sewage will be accommodated by existing or enlarged conventional treatment facilities. For separated storm water runoff or for combined flows it was presumed that the minimum acceptable treatment under any circumstances will include chlorination, even though this has not been classified as a candidate system.

Dissolved Air Flotation

To provide a common basis for comparison of dissolved air flotation with other alternative treatment processes, a unit output capacity of 40 mgd was established. Total system requirements were met with modules of this basic unit. Two dissolved air flotation preliminary designs were prepared, one for the condition of 30-percent removal of suspended solids and the other for 60-percent removal.

A conceptual plan of a 40-mgd unit for 30-percent suspended solids removal is shown in Figure 50. Ideally, a reservoir preceding the treatment process would provide a controlled inflow. A minimum water level within the basin would be attained prior to starting the treatment facility. If this minimum level were not attained before runoff has subsided and trunk line capacity became available, the impounded water would be pumped back to the trunk line at a rate not to exceed existing treatment plant capacity. If the water level continued to rise, the treatment facility would be started and flows processed until the basin has been emptied.

Plant start-up may be completely automated from the time the process comes on the line until it is shut down. Assuming controlled inflow, approximately 2.6 hours would be required to fill the tanks and for the operation to function fully. On a level signal, the skimmers and collectors would be started along with the effluent recirculation pumps and air compressor. During the initial phase of operation, prechlorination may be desirable to minimize odors.

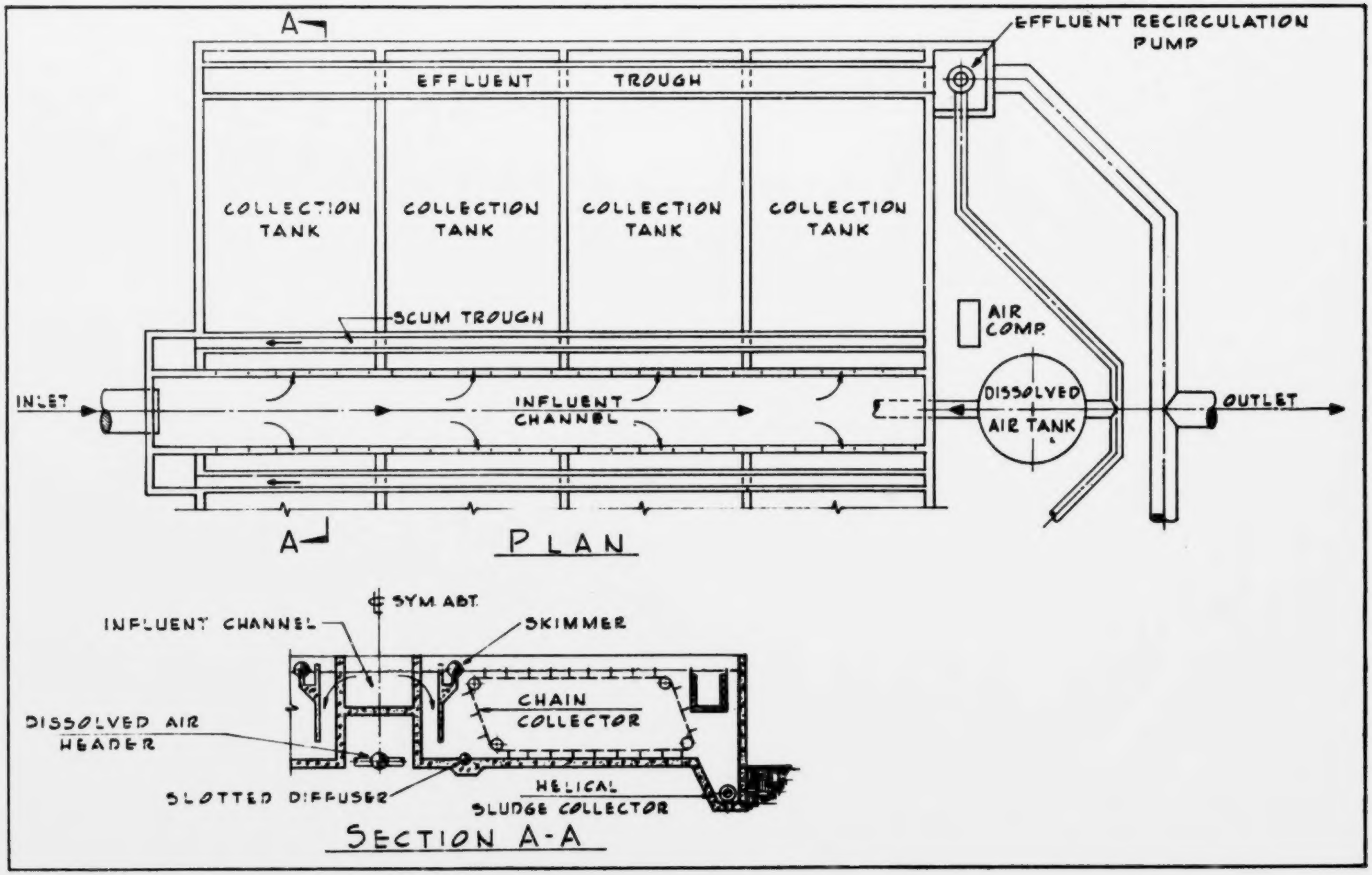

A
EFFLUENT RECIRCULATION PUMP
EFFLUENT TROUGH
COLLECTION TANK
COLLECTION TANK
COLLECTION TANK
COLLECTION TANK
SCUM TROUGH
AIR COMP.
INLET
INFLUENT CHANNEL
DISSOLVED AIR TANK
OUTLET
A
PLAN
℄ SYM. ABT.
INFLUENT CHANNEL
SKIMMER
CHAIN COLLECTOR
DISSOLVED AIR HEADER
SLOTTED DIFFUSER
HELICAL SLUDGE COLLECTOR
SECTION A-A

Figure 50. DISSOLVED AIR FLOTATION UNIT

Incoming wastewater would be distributed via a channel into each tank. Within the tank, the flow would be diverted downward and across the dissolved air header wherein effluent saturated with oxygen is to be introduced. The air-saturated effluent would carry floatables to the surface and dense particulates would settle to the bottom. The floatables on the tank water surface and the settled matter on the bottom would be transported to a rotating skimmer and a helical sludge collector, respectively, by traveling chain collectors. The floatables would travel to a common pit for return to the trunk sewer together with the sludge withdrawn from the tank hoppers. Each hopper and pit would be pumped individually through air-actuated valves rotationally sequenced by means of a timer.

Plant effluent would pass upward beyond a baffle into a quiescent area and flow over weirs into the effluent channel. The recirculation pumps would return 20 percent of the flow to the tanks at a pressure of 65 psig. Compressed air at 30 cfm would be injected into the pipeline prior to a dissolved air tank with a retention period of one minute. From the dissolved air tank, the saturated effluent would be fed to the slotted tank headers through a distribution main.

At the termination of operation, the plant effluent may be recirculated to wash down the storage reservoir and pump station. Final dewatering of the plant may be accomplished by using the sludge and recirculation pumps.

Mechanical Screening

The SWECO mechanical screening unit applied here has been termed the Wastewater Concentrator and was designed primarily for the removal of substantial portions of both floatable and settleable solids, but it also has the capability of removing from 32 to 36 percent of the suspended solids and achieving BOD reductions of from 12 to 23 percent. The flow split between treated effluent and concentrate used in this program was approximately 80:20, although very recent data from the manufacturer would indicate that similar removals of solids can be effected at a split of 88:12.

The unit consists of a cylindrical screen rotating about a vertical axis. The feedwater enters through a central inlet pipe and flows horizontally outward to the revolving screen over a distribution dome. The cylindrical screen revolves at between 50 to 60 rpm and passes approximately 80 to 88 percent of the flow out of the unit as effluent. The remaining 12 to 20 percent drops down the inside of the screen and is discharged separately from the unit as concentrate.

Each separator unit employs an inside-outside hot water spray cleaning system operated by cycle timers. Best performance reportedly occurs with 4.5-minute on and 0.5-minute off cycles. During the on cycle, the outside hot water spray system operates. At the end of

4.5 minutes, the feed flow is interrupted and the back-spray system then operates for 30 seconds.

Each separator unit is powered by a 5-hp, 220/440-v, 3-phase motor. The screens consist of a series of segmented, easily replaceable panels which, in the case of the 8-ft diameter unit envisioned for the 40-mgd module, would contain approximately 24 panel screens. Periodic inspection and replacement of screens would be required.

For the Sacramento application, fourteen 8-ft diameter separators were combined within a common effluent basin shown in Figure 51. The 14-unit module would give a net effluent outflow rate of 40 mgd, requiring an influent rate of 50 mgd with the 80:20 flow split.

In the 40-mgd module the system would be activated by liquid-level controls, which would turn on the hot water system, the influent pumps, and the module control system. The module control system would energize the screen drive motors and activate the air-operated butterfly valves to the individual units in a sequential order. Timers would activate the solenoid valves in the spray lines inside and outside the cylindrical screen. Shutdown operations would follow a similar pattern in reverse.

In considering overall study area requirements, the 20-percent volume of concentrate produced by the separator represents a significant volume still requiring treatment and disposal. Since this concentrate flow will frequently exceed the off-peak capacity of the treatment plant, the screening process was considered feasible only in combination with an upstream reservoir with provisions made for recycling of the concentrate back through the reservoir. Thus the net 80-percent effluent flow rate would be maintained continually and the concentrate treatment accomplished through controlled input to the treatment plant as off-peak capacity became available. As discussed later under Trade-Off Optimization, the inclusion of the reservoir in combination with the screening process did not impose an undue burden on the subsystem, since a cost savings was also achieved.

During the preparation of this report, the manufacturer submitted new data showing significantly improved performance for the screening units. The improved performance was derived from a flow split ratio of 88 parts effluent to 12 parts concentrate rather than the previously reported 80:20, and from a greater input flow through the standard 5-ft diameter model than previously considered feasible. For the 40-mgd module, sixteen 5-ft diameter separator units are indicated rather than the previously considered fourteen 8-ft diameter separator units.

COSTS

Dissolved air flotation treatment costs were developed for systems capable of suspended solids reductions of 30 and 60 percent. Mechanical screening treatment costs were developed for a system effecting

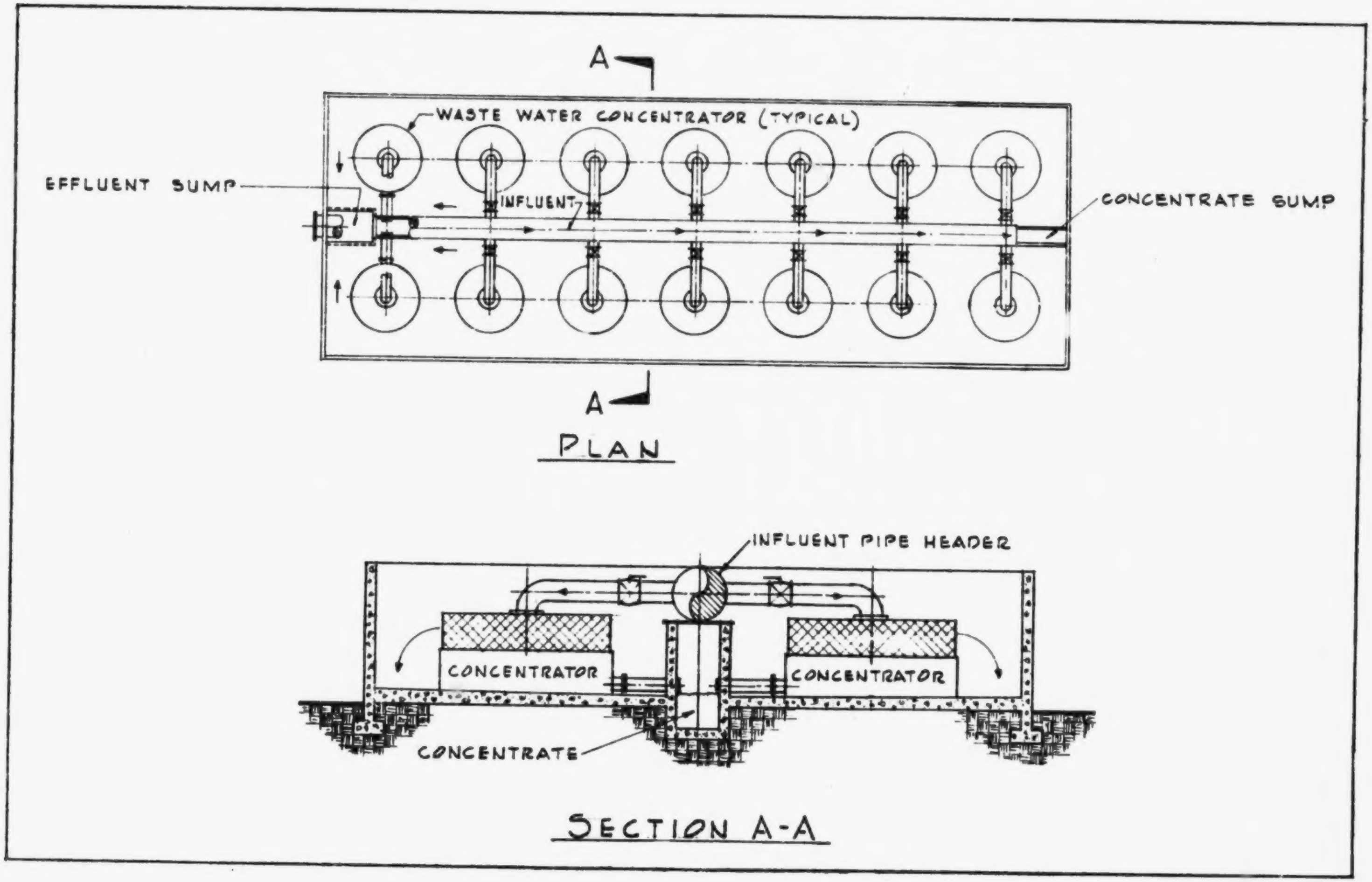

A
WASTE WATER CONCENTRATOR (TYPICAL)
EFFLUENT SUMP
INFLUENT
CONCENTRATE SUMP
A
PLAN
INFLUENT PIPE HEADER
CONCENTRATOR
CONCENTRATOR
CONCENTRATE
SECTION A-A

Figure 51. MECHANICAL SCREENING UNIT

30-percent removal of suspended solids.

Multiples of the basic 40-mgd modules were applied as required to meet total flow requirements as specified either by peak conditions or reservoir withdrawal rates. The total annual costs are summarized in Table 20 for the several locations under consideration. An annual operation and maintenance cost allowance is included. There are no associated pumping or connecting piping costs lumped with treatment costs that remain a requirement independently of the treatment process. Therefore, as shown in Table 20, the alternative of no treatment produced a zero cost value.

The costs presented in Table 20 for the mechanical screening process do not reflect the most recent performance data, which were received too late for processing. It is estimated that the incorporation of this improved performance data would effect a cost reduction of about 25 percent below the values presented. Therefore, for 30-percent removals, the mechanical screening process should prove less costly than dissolved air flotation.

COMPARISON

Comparison of 30- and 60-percent removals for the dissolved air flotation process becomes a matter of relating resultant effluent quality to cost, which will be considered later.

For 30-percent removals, a comparison between dissolved air flotation and mechanical screening is difficult because of the general lack of quantitative data. For both alternatives, additional testing under field operating conditions would be desirable before proceeding with the design and construction of a full-scale system. While not reflected in Table 20, the mechanical screening process would be selected on the basis of cost alone if the improved performance data were considered.

EFFLUENT DISPOSAL

Effluent disposal has been included as a major subsystem classification in order that each of the candidate systems may be evaluated within the framework of a complete sewer system. Effluent disposal requirements were varied only to the extent necessary to satisfy the complete sewer system criteria as the other alternatives were manipulated. Accordingly, effluent disposal was not in itself considered as an alternative.

DESCRIPTION

Chlorination was established as a mandatory requirement prior to effluent disposal, regardless of any preceding treatment process that may be specified. Depending on the location of treatment facilities, a contact chamber of sufficient capacity was provided such that total elapsed time prior to river discharge is a minimum of 15 minutes subsequent to

Table 20

ANNUAL COSTS OF TREATMENT FOR VARIED INFLUENT FLOW RATES
(millions of dollars)

Location	Influent Flow Rate, mgd	Air Flotation 30% Suspended Solids Removal				Air Flotation 60% Suspended Solids Removal				Mechanical Screening 30% Suspended Solids Removal			
		Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow	Storm Flow	Combined Flow
American River Park		Separated Area				Separated Area				Separated Area			
Surface Reservoir	10	-				-				-			
	20	.09				.15				.12			
	40	.16				.27				.21			
	80	.29				.50				.39			
	160	.53				.92				.69			
	231*	.54				1.08				-			
	320	1.09				1.83				1.26			
	352*	.79				1.57				-			
	422*	.93				1.85				-			
Sump 2 Vicinity		Combined Area		Study Area		Combined Area		Study Area		Combined Area		Study Area	
	40	.11	.12	.12	.13	.21	.22	.23	.25	.14	.14	.15	.15
Underground Reservoir	80	.21	.21	.22	.23	.41	.42	.43	.45	.26	.27	.28	.29
	160	.38	.39	.40	.42	.75	.77	.79	.83	.48	.49	.51	.53
	320	.71	.73	.75	.79	1.41	1.45	1.49	1.57	.89	.91	.94	.98
	575*	1.22	-	-	-	2.45	-	-	-	-	-	-	-
	640	1.29	1.33	1.38	1.45	2.55	2.63	2.73	2.87	1.65	1.70	1.76	1.85
	842*	-	1.73	-	-	-	3.46	-	-	-	-	-	-
	892*	1.82	-	-	-	3.64	-	-	-	-	-	-	-
	993*	-	2.00	-	-	-	4.01	-	-	-	-	-	-
	1077*	2.16	-	-	-	4.32	-	-	-	-	-	-	-
Sacramento Port		Combined Area		Study Area		Combined Area		Study Area		Combined Area		Study Area	
	-	-	-	-	-	-	-	-	-	-	-	-	-
Surface Reservoir	40	.12	.13	.13	.14	22	.23	.24	.26	.21	.22	.22	.23
	80	.28	.29	.30	.32	.47	.49	.51	.55	.39	.39	.40	.41
	160	.50	.52	.53	.56	.86	.90	.92	.98	.68	.69	.70	.72
	320	.93	.96	.98	1.03	1.62	1.68	1.72	1.82	1.23	1.25	1.27	1.31
	640	1.70	1.74	1.78	1.86	2.97	3.05	3.13	3.29	2.23	2.27	2.32	2.42
South Sacramento		Combined Area		Study Area		Combined Area		Study Area		Combined Area		Study Area	
	-	-	-	-	-	-	-	-	-	-	-	-	-
Surface Reservoir	40	.15	.16	.16	.17	.25	.26	.27	.29	.19	.20	.20	.21
	80	.27	.28	.29	.31	.46	.48	.50	.54	.36	.36	.37	.38
	160	.49	.51	.52	.55	.85	.89	.91	.97	.66	.67	.68	.70
	320	.93	.96	.98	1.03	1.62	1.68	1.71	1.81	1.20	1.22	1.24	1.28
	640	1.70	1.74	1.78	1.86	2.97	3.05	3.13	3.29	2.20	2.24	2.29	2.39

* Peak Flow (No Reservoir)

30-percent removal of suspended solids.

Multiples of the basic 40-mgd modules were applied as required to meet total flow requirements as specified either by peak conditions or reservoir withdrawal rates. The total annual costs are summarized in Table 20 for the several locations under consideration. An annual operation and maintenance cost allowance is included. There are no associated pumping or connecting piping costs lumped with treatment costs that remain a requirement independently of the treatment process. Therefore, as shown in Table 20, the alternative of no treatment produced a zero cost value.

The costs presented in Table 20 for the mechanical screening process do not reflect the most recent performance data, which were received too late for processing. It is estimated that the incorporation of this improved performance data would effect a cost reduction of about 25 percent below the values presented. Therefore, for 30-percent removals, the mechanical screening process should prove less costly than dissolved air flotation.

COMPARISON

Comparison of 30- and 60-percent removals for the dissolved air flotation process becomes a matter of relating resultant effluent quality to cost, which will be considered later.

For 30-percent removals, a comparison between dissolved air flotation and mechanical screening is difficult because of the general lack of quantitative data. For both alternatives, additional testing under field operating conditions would be desirable before proceeding with the design and construction of a full-scale system. While not reflected in Table 20, the mechanical screening process would be selected on the basis of cost alone if the improved performance data were considered.

EFFLUENT DISPOSAL

Effluent disposal has been included as a major subsystem classification in order that each of the candidate systems may be evaluated within the framework of a complete sewer system. Effluent disposal requirements were varied only to the extent necessary to satisfy the complete sewer system criteria as the other alternatives were manipulated. Accordingly, effluent disposal was not in itself considered as an alternative.

DESCRIPTION

Chlorination was established as a mandatory requirement prior to effluent disposal, regardless of any preceding treatment process that may be specified. Depending on the location of treatment facilities, a contact chamber of sufficient capacity was provided such that total elapsed time prior to river discharge is a minimum of 15 minutes subsequent to chlorination. A typical chlorination facility is shown in Figure 52. Where peak flows without benefit of storage are to be handled, the 15-minute elapsed time was reduced to ten minutes for the peak flow condition. In those cases where storage without subsequent treatment was specified, the reservoir itself would function as the contact chamber. The outfall sewer line requirements and associated lift station requirements are also included under the effluent disposal category.

COSTS

The annual costs, including operation and maintenance costs, relating to effluent disposal are summarized in Table 21.

TRADE-OFF OPTIMIZATION

Wherever storage was included as a part of a complete sewer system, the specified withdrawal rate effected not only reservoir size but also downstream treatment and effluent disposal sizing requirements. Functioning as a peak-flow modulator, the reservoir size and cost increases as the withdrawal rate is reduced, while downstream the facilities capacity requirements and costs decrease.

Ignoring resultant effluent wastewater quality variations, an optimum (least cost) arrangement of capacities was established for all alternatives within the major subsystem classifications of storage, treatment, and effluent disposal. The resulting reservoir sizes, withdrawal rates, and combined storage-treatment-disposal minimum costs are summarized in Table 22.

A tabular listing of alternatives by major subsystems is given in Table 23, which includes an identification letter and number for keying into Tables 24 through 30, which present the summary costs of the complete sewer systems for the Study Area.

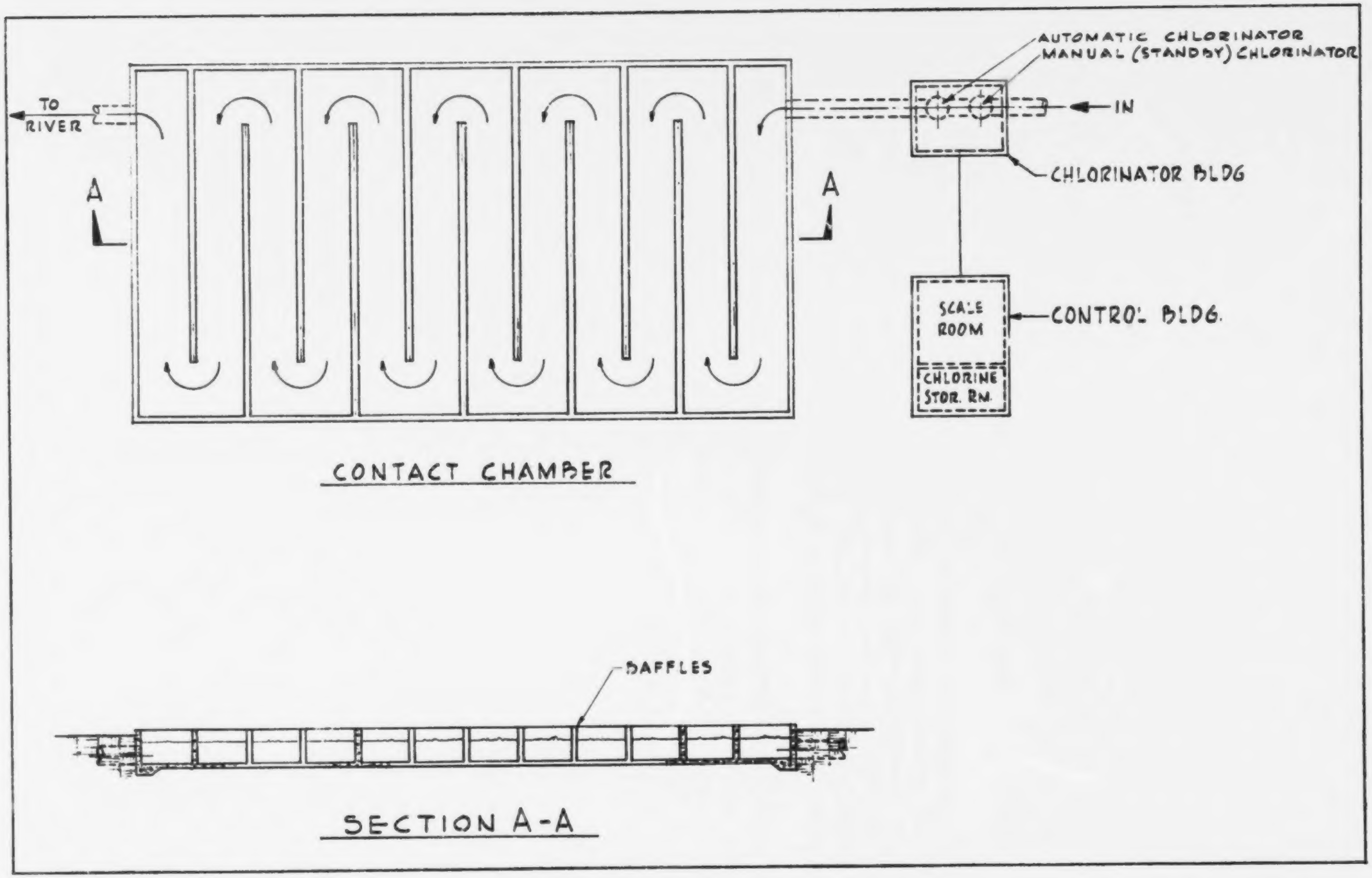

AUTOMATIC CHLORINATOR
MANUAL (STANDBY) CHLORINATOR
TO RIVER
IN
A
A
CHLORINATOR BLDG
SCALE ROOM
CHLORINE STOR. RM.
CONTROL BLDG.
CONTACT CHAMBER
BAFFLES
SECTION A-A

Figure 52. CHLORINATION FACILITY

Table 21

ANNUAL COSTS OF EFFLUENT CHLORINATION AND DISPOSAL

Location	Storm Recurrence Expectancy, years	Peak Storm Flow, mgd	Peak Combined Flow, mgd	Reservoir Withdrawal Rate, mgd	Annual Cost, million $
American River Park	-	-	-	20	0.03
	-	-	-	40	0.04
Separated Area	-	-	-	80	0.05
Surface Reservoir	-	-	-	160	0.07
	-	-	-	320	0.10
	2	231*	-	-	0.07
	5	352*	-	-	0.09
	10	422*	-	-	0.10
Sump 2 Vicinity	-	-	-	40	0.04
	-	-	-	80	0.05
Combined Area	-	-	-	160	0.07
Underground	-	-	-	320	0.22
Reservoir	-	-	-	640	0.48
	2	577*	-	-	0.35
	5	892*	-	-	0.48
	10	1077*	-	-	0.55
	2	-	575*	-	0.38
	5	-	842*	-	0.51
	10	-	993*	-	0.58
Sacramento Port	-	-	-	10	0.03
	-	-	-	40	0.04
Combined Area	-	-	-	80	0.06
Surface Reservoir	-	-	-	160	0.08
	-	-	-	320	0.11
	-	-	-	640	0.16
South Sacramento	-	-	-	10	0.09
	-	-	-	40	0.14
Combined Area	-	-	-	80	0.06
Surface Reservoir	-	-	-	160	0.31
	-	-	-	320	0.52
	-	-	-	640	0.90

*No reservoir

Table 22

COMBINATIONS OF STORAGE, TREATMENT, AND DISPOSAL RESULTING IN MINIMUM ANNUAL COSTS

Location	Treatment and Percent Removal of Suspended Solids	2-yr Storm Recurrence								5-yr Storm Recurrence								10-yr Storm Recurrence							
		Storm Flow		Combined Flow		Storm Flow		Combined Flow		Storm Flow		Combined Flow		Storm Flow		Combined Flow		Storm Flow		Combined Flow		Storm Flow		Combined Flow	
		Withdrawal Rate, mgd	Annual Cost, million $	Withdrawal Rate, mgd	Annual Cost, million $	Withdrawal Rate, mgd	Annual Cost, million $	Withdrawal Rate, mgd	Annual Cost, million $	Withdrawal Rate, mgd	Annual Cost, million $	Withdrawal Rate, mgd	Annual Cost, million $	Withdrawal Rate, mgd	Annual Cost, million $	Withdrawal Rate, mgd	Annual Cost, million $	Withdrawal Rate, mgd	Annual Cost, million $	Withdrawal Rate, mgd	Annual Cost, million $	Withdrawal Rate, mgd	Annual Cost, million $	Withdrawal Rate, mgd	Annual Cost, million $
American River Park Surface Reservoir		Separated Area								Separated Area								Separated Area							
	Air Flotation, 30%	40	1.13							50	1.61							50	1.87						
	Air Flotation, 60%	20	1.21							30	1.72							40	1.99						
	Mech. Screens, 30%	30	1.17							40	1.67							45	1.92						
Sump 2 Underground Reservoir		Combined Area				Study Area				Combined Area				Study Area				Combined Area				Study Area			
	Air Flotation, 30%	220	1.65	260	1.85	280	3.32	340	3.52	300	2.02	320	2.20	400	4.19	360	4.23	270	2.24	340	2.44	360	4.81	400	5.02
	Air Flotation, 60%	160	2.10	200	2.38	200	3.86	280	4.22	190	2.55	240	2.83	260	4.92	320	4.99	200	2.79	240	3.11	260	5.50	320	5.82
	Mech. Screens, 30%	220	1.80	240	2.01	240	3.48	320	3.73	260	2.19	280	2.40	300	4.42	340	4.45	240	2.40	300	2.64	320	5.02	360	5.25
Sacramento Port Surface Reservoir		Combined Area				Study Area				Combined Area				Study Area				Combined Area				Study Area			
	Air Flotation, 30%	80	1.29	140	1.47	140	2.80	180	2.98	140	1.75	180	1.84	180	3.63	260	3.87	180	1.94	180	2.00	180	4.14	260	4.52
	Air Flotation, 60%	40	1.46	100	1.78	80	3.06	110	3.31	80	2.02	110	2.23	140	4.03	140	4.38	110	2.30	140	2.38	140	4.52	180	5.05
	Mech. Screens, 30%	60	1.39	120	1.62	120	2.94	120	3.14	120	1.89	160	2.03	160	3.83	240	4.12	140	2.12	160	2.19	150	4.33	240	4.77
South Sacramento Surface Reservoir		Combined Area				Study Area				Combined Area				Study Area				Combined Area				Study Area			
	Air Flotation, 30%	70	2.32	120	2.59	120	3.99	120	4.41	100	3.17	140	3.35	140	5.34	210	5.82	140	3.63	140	3.63	160	6.12	210	7.02
	Air Flotation, 60%	50	2.50	80	2.87	60	4.21	110	4.72	60	3.39	120	3.66	110	5.67	100	6.17	80	3.93	120	3.95	140	6.49	140	7.44
	Mech. Screens, 30%	60	2.40	120	2.71	80	4.08	120	4.53	80	3.26	120	3.48	140	5.48	140	5.99	120	3.76	140	3.76	150	6.28	190	7.20

Table 23

KEY FOR COMPLETE SEWER SYSTEM COSTING COMBINATIONS

(Presented in Tables 24 through 30)

A - SEPARATED AREA

1. Treatment at Individual Sumps
2. Treatment at American River Park
3. Storm Flow to Sump No. 2

B - CONVEYANCE SYSTEM

1. New Gravity Storm Sewers (Alternate 1)
2. Augmentation of Existing Combined Sewers Pipe-in-Pipe/New Gravity Storm Sewers Upstream (Alternate 3)
3. Hybrid Force Main (Alternate 7)
4. Hybrid Force Main - New Gravity Storm Sewers (Alternate 5)
5. Existing Storm System Augmented

C - STORAGE

1. Underground - Storm Runoff
2. Underground - Combined Sewage
3. No Reservoir
4. Surface - Storm Runoff
5. Surface - Combined Sewage
6. Stabilization Pond - Storm Runoff
7. Stabilization Pond - Combined Sewage

D - TREATMENT AND DISPOSAL

1. Air Flotation - 30% Suspended Solids Removal
2. Air Flotation - 60% Suspended Solids Removal
3. Mechanical Screening - 30% Suspended Solids Removal
4. No Treatment (Except Chlorination)
5. Conventional Sewage Treatment (40 mgd)

Table 24

ANNUAL SYSTEM COSTS, SEPARATED AREA*
(millions of dollars)

				Storm Recurrence		
A	B	C	D	2-yr	5-yr	10-yr
1	5	3	4	0.65	0.96	1.07
2	5	4	1	1.43	2.10	2.39
2	5	3	1	1.68	2.53	2.93
2	5	4	2	1.51	2.21	2.50
2	5	3	2	2.22	3.31	3.86
2	5	6	4	1.50	2.23	2.60
2	5	3	4	1.14	1.74	2.01

*Reference Table 23 Key

Table 25

ANNUAL SYSTEM COSTS, COMBINED AREA SUMP NO. 2 VICINITY LOCATION*
(millions of dollars)

				Storm Recurrence		
A	B	C	D	2-yr	5-yr	10-yr
	1	1	1	4.03	5.21	5.88
	1	1	2	4.48	5.74	6.43
	1	1	3	4.18	5.39	6.04
	1	3	4	2.78	3.73	4.28
	1	3	1	4.34	6.06	7.03
	1	3	2	5.57	7.88	9.19
	2	2	1	3.71	4.81	5.40
	2	2	2	4.24	5.44	6.07
	2	2	3	3.87	5.00	5.60
	2	3	1	3.86	5.39	6.16
	2	3	2	5.09	6.61	8.17
	3	1	1	6.05	7.45	8.06
	3	1	2	6.51	7.98	8.62
	3	1	3	6.20	7.62	8.23
	3	3	4	4.80	5.97	6.47
	3	3	1	6.36	8.30	9.22
	3	3	2	7.59	10.12	11.38
	4	1	1	15.14	16.09	16.63
	4	1	2	15.59	16.62	17.18
	4	1	3	15.29	16.27	16.79
	4	3	4	13.89	14.61	15.03
	4	3	1	15.45	16.94	17.78
	4	3	2	16.67	19.76	19.94
	2	3	4	2.29	3.18	3.62

*Reference Table 23 Key

Table 26

ANNUAL SYSTEM COSTS, COMBINED AREA SACRAMENTO PORT LOCATION*

(millions of dollars)

				Storm Recurrence		
A	B	C	D	2-yr	5-yr	10-yr
	1	4	1	3.67	4.94	5.58
	1	4	2	3.84	5.22	5.94
	1	4	3	3.77	5.09	5.76
	1	6	4	3.85	5.29	6.06
	2	5	1	3.33	4.45	4.96
	2	5	2	3.64	4.84	5.34
	2	5	3	3.48	4.64	5.15
	2	7	4	4.26	6.01	6.92
	3	4	1	5.69	7.18	7.77
	3	4	2	5.86	7.45	8.13
	3	4	3	5.79	7.33	7.95
	3	6	4	5.87	7.52	8.24
	4	4	1	14.77	15.82	16.33
	4	4	2	14.94	16.10	16.69
	4	4	3	14.87	15.97	16.51
	4	6	4	14.95	16.17	16.81
	1	4	5	4.07	5.65	6.46
	2	5	5	4.10	5.78	6.56
	3	4	5	6.09	7.89	8.65
	4	4	5	15.17	16.53	17.21

*Reference Table 23 Key

Table 27

ANNUAL SYSTEM COSTS, COMBINED AREA SOUTH SACRAMENTO LOCATION*

(millions of dollars)

				Storm Recurrence		
A	B	C	D	2-yr	5-yr	10-yr
	1	4	1	4.70	6.37	7.28
	1	4	2	4.88	6.58	7.57
	1	4	3	4.78	6.46	7.40
	1	6	4	4.80	6.60	7.56
	1	4	5	4.89	6.83	7.83
	2	5	1	4.45	5.95	6.58
	2	5	2	4.73	6.27	6.91
	2	5	3	4.57	6.09	6.71
	2	7	4	5.22	7.33	8.34
	2	5	5	4.93	6.96	7.84
	3	4	1	6.72	8.60	9.46
	3	4	2	6.90	8.82	9.76
	3	4	3	6.80	8.70	9.59
	3	6	4	6.83	8.84	9.75
	3	4	5	6.91	9.07	10.02
	4	4	1	15.80	17.25	18.03
	4	4	2	15.98	17.46	18.32
	4	4	3	15.88	17.34	18.15
	4	6	4	15.91	17.48	18.31
	4	4	5	15.99	17.71	18.58

*Reference Table 23 Key

Table 28

ANNUAL SYSTEM COSTS, STUDY AREA SUMP NO. 2 VICINITY LOCATION*

(millions of dollars)

A	B	C	D	Storm Recurrence 2-yr	5-yr	10-yr
3	1	1	1	6.00	7.87	8.97
3	1	1	2	6.54	8.61	9.67
3	1	1	3	6.16	8.10	9.18
3	2	2	1	5.68	7.33	8.50
3	2	2	2	6.38	8.09	9.30
3	2	2	3	5.88	7.55	8.73
3	3	1	1	8.02	10.11	11.16
3	3	1	2	8.56	10.84	11.85
3	3	1	3	8.18	10.34	11.37
3	4	1	1	17.10	18.75	19.72
3	4	1	2	17.65	19.49	20.42
3	4	1	3	17.27	18.98	19.93

*Reference Table 23 Key

Table 29

ANNUAL SYSTEM COSTS, STUDY AREA
SACRAMENTO PORT LOCATION*

(millions of dollars)

				Storm Recurrence		
A	B	C	D	2-yr	5-yr	10-yr
3	1	4	1	5.48	7.32	8.20
3	1	4	2	5.75	7.71	8.69
3	1	4	3	5.62	7.51	8.50
3	2	5	1	5.14	6.97	8.00
3	2	5	2	5.47	7.48	8.53
3	2	5	3	5.30	7.22	8.25
3	3	4	1	7.50	9.56	10.49
3	3	4	2	7.76	9.95	10.87
3	3	4	3	7.64	9.75	10.68
3	4	4	1	16.58	18.20	19.05
3	4	4	2	16.85	18.59	19.44
3	4	4	3	16.72	18.39	19.25
3	1	4	5	4.62	6.32	7.19
3	2	5	5	5.25	7.21	8.28
3	3	4	5	6.64	7.56	9.38
3	4	4	5	15.72	17.20	17.94

*Reference Table 23 Key

Table 30

ANNUAL SYSTEM COSTS, STUDY AREA SOUTH SACRAMENTO LOCATION*

(millions of dollars)

					Storm Recurrence	
A	B	C	D	2-yr	5-yr	10-yr
3	1	4	1	6.67	8.93	10.29
3	1	4	2	6.89	9.35	10.65
3	1	4	3	6.76	9.17	10.44
3	1	4	5	5.54	7.68	8.83
3	2	5	1	6.57	8.92	10.50
3	2	5	2	6.88	9.27	10.92
3	2	5	3	6.60	9.09	10.68
3	2	5	5	6.38	8.74	10.35
3	3	4	1	8.69	11.26	12.47
3	3	4	2	8.91	11.59	12.84
3	3	4	3	8.79	11.40	12.62
3	3	4	5	7.56	9.92	11.02
3	4	4	1	17.77	19.91	21.04
3	4	4	2	17.99	20.23	21.40
3	4	4	3	17.87	20.05	21.19
3	4	4	5	16.64	18.56	19.58

*Reference Table 23 Key

Section VIII

SYSTEMS EVALUATION AND SELECTION

The evaluation of systems for the control of water pollution from storm water runoff and combined sewer overflows for the purpose of presenting one or more systems for selection depends upon several inter-related factors. The major considerations are system performance, system cost, and other system effects on the environment in regard to meeting acceptable and specified standards or criteria. Actual selection of the system must be made by the administrators responsible for their implementation, financing, and operation and will depend upon many complex political, social, and economic considerations that are beyond the usual rigorous definition, quantification, and analysis. Thus the best that technology can do is present the costs, performance, and effects of alternative public works systems in a display that will aid the administrator in his selection.

EVALUATION CRITERIA

The adequacy of any alternative system must be measured against criteria that have been established to allow the attainment of public goals and objectives. The criteria must be realistic in magnitude and form to permit implementation and operation within a community's constraints of economics, responsibilities, and attitudes.

SYSTEM PERFORMANCE

The systems considered in this program should be capable of performing two major functions designed to eliminate or significantly reduce detrimental effects on the environment and to decrease any impairment to public well-being and enjoyment.

The first is the management of storm water runoff and combined sewer overflow quantities so as to reduce the occurrence of or eliminate uncontrolled discharges from treatment facilities to the receiving waters. In this program, all alternative systems were designed to provide the same adequate hydraulic capacity for three storm recurrence expectancies. Thus, all alternative systems designed for a particular storm recurrence interval perform comparably and require no evaluation with regard to their hydraulic performance.

The second performance function has to do with the effect of the alternative system on the quality of the environment. Whereas the hydraulic aspects are local in impact, the system's performance in regard to water quality must be judged from the viewpoint of not only local but regional and national considerations of the total water resource management system. The method developed in this program assesses the extent of the pollution in the immediate vicinity occurring as the result of alternative storm water runoff and combined sewage control systems. While no attempt has been made to extend the approach to consider the spatial and temporal changes resulting downstream from the installation and operation of the various alternatives, this expansion in the methodology could be accomplished. However, those considerations depend quite heavily upon the type of receiving water system, i.e., river, lake, embayment, estuary, or ocean.

Hydraulic Considerations

Most structures for the containment of naturally occurring waters are designed on the basis of the average occurrence or expectancy of single events, which have associated with them a particular expected magnitude. Thus hydraulic works are sized to handle at least the quantities of water associated with storms that are lesser in magnitude than the design storm of a particular recurrence interval. Large structures, such as dams and reservoirs built for controlling floods are designed to withstand and contain severe storms that occur on the average once every 50 or 100 years because their inadequacy could result in major destruction of life and property. Structures for controlling lesser quantities of storm runoff are sized for correspondingly lesser storm intensities and duration, which occur more frequently; flood control channels are adequate to contain the runoff from storms that recur once every 25 or 50 years. Buried conduit or sewer systems on the other hand are usually designed to carry adequately only the runoff from storms that can be expected to occur on the average once every five to ten years. The relatively frequent inadequacy of these systems can be justified on the grounds that during the deficient periods little if any loss of life or property occurs, that the system is usually conservative in design and capable of somewhat greater capacity during periods of surcharging and minor flooding of overlying surfaces, and that the inconveniences caused are tolerable and acceptable to the public.

In this analysis, conveyance systems and their hydraulically related appurtenances were designed to have adequate capacity for all expected storms that recur only once every two years, once every five years, and once every ten years. The three recurrence intervals were chosen for several reasons. First, it was not known a priori what storm interval best represented the existing conveyance system, although recent separation projects were based reportedly on 10-year recurrence intervals. The correct recurrence expectancy was important for establishing requirements in supplementing the existing system to bring it to the arbitrarily selected design capacity. Secondly, since all three could be readily and rapidly accommodated in the design procedure, it would permit a comparison of costs between the various systems for the three return intervals.

All further discussion herein is based on systems designed to perform adequately without surcharging for storms of severity less than one that would be expected to occur on the average once every five years. It is believed that systems designed to this storm recurrence interval for the conditions in the Sacramento Study Area are adequate and realistic, based primarily on the apparent public acceptance of a system which by the design standards and procedures employed in this program will not be completely adequate for a storm of 2-year recurrence.

Water Quality Aspects

The performance of facilities in achieving acceptable water quality control objectives can be established on the basis of requirements applied either to the waste discharge quality or to the receiving water quality following discharge of the wastes and their immediate assimilation. In many cases and certainly for the pollutants considered pertinent to this program, requirements placed directly on wastewater discharges must be based on a careful determination and evaluation of the assimilative capacity of receiving waters and the effects on overall water quality. Because the natural quality of the receiving waters for the storm water runoff and combined sewer overflows from the Study Area were quite variable over a rather large range and were flow dependent, it was imperative in this program to consider the extent to which the various alternative systems affected the receiving water quality following the admixing of the wastewater discharges, particularly since some of the wastewater pollutant characteristics were somewhat similar in magnitude and the quantities of discharge were usually quite small compared to those of the receiving waters.

Due to the variations in wastewater and receiving water quality and quantity, it is not possible to establish realistic single values or sets of water quality criteria that will suffice for all occasions at all times. Instead, a distribution of wastewater and receiving water qualities exists that represents, on the average, both naturally occurring and humanly controlled systems, and the specified criteria should reflect these distributions and the associated expectancies. Three water quality criterion parameters are suggested to completely determine the ability of alternative systems to perform adequately in meeting water quality objectives. A system, to be acceptable, must meet all three criteria.

The first water quality criterion parameter establishes an absolute maximum concentration of pullutant that cannot be exceeded at any time. This parameter will prevent the occurrence of acute lethal dosages of pollutants for the protection of the public health and the fisheries resource and other single, but catostrophic, occurrences. The second parameter establishes an acceptable distribution of pollutant concentrations by specifying a frequency of occurrence of a particular value. That is to say that a specified value can be exceeded only a certain small percentage of the total time. This parameter recognizes the presence of periods in which the receiving waters are below acceptable quality which does not constitute long-term degradation but only tolerable transient conditions. All uses of the receiving waters dependent upon pollutant concentration-time relations such as chronic animal toxicity are accommodated by this criterion parameter. In those downstream water uses however that require a change or stoppage of procedure during those periods when pollutant concentrations are excessive, such as could occur for water supply operations, the total duration of the occurrence of excessive concentration is probably of less concern than the number of times that the value is exceeded and extraordinary operational procedures initiated. This is similar to the determination of hydraulic capacities based on recurrence interval of a particular storm and the associated runoff volumes, where the number of flooding episodes or the meer existance of flooding is of more importance than the total duration of flooding. Whereas a cumulative frequency distribution parameter indicates the total time over which any particular value is exceeded, e.g., 12 hours annually, it does not indicate whether this value will be exceeded once for a total of 12 hours each year or on 12 separate occasions each of 1-hour duration. The third water quality criterion parameter accounts for this by establishing the acceptable number of occurrences or expectancy for any particular concentration to be exceeded.

Therefore it is proposed that these criterion parameters, all of which must be satisfied, can adequately and completely describe the required performance of a water quality control facility. These will be termed maximum value criterion, cumulative distribution criterion, and excession frequency criterion.

As will be discussed subsequently, if the pertinent water quality characteristics of the receiving water are unacceptably or undesirably high as the result of uncontrolled surface runoff or upstream discharges of inadequately treated wastewaters and the relative mixing volume of receiving water quite great, the evaluation of alternative systems is extremely difficult since the discharge of a highly treated wastewater has no measurable effect different from the discharge of the same influent wastewater untreated. Since this situation occurs in this program with regard to suspended solids and fecal coliforms the performance of the alternative systems in controlling these pollutants will be judged differently from that used for BOD, which was based on simulated discharge to the receiving water of anticipated indigenous quality.*

The system performance for controlling the discharge of suspended solids was based on the incremental addition of suspended solids to the receiving waters by considering that the receiving waters contained no suspended solids or impurities prior to discharge of the treated wastewaters. In this way the contribution of various alternative systems to the total receiving water burden could better be assessed.

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*In effect, the same holds true for BOD, since a constant low-level value, derived from expected likely future levels, was assumed to obtain at all times.

Fecal coliforms are not included in subsequent discussions, since chlorination facilities capable of virtually total disinfection were provided in all alternative systems, including the no treatment configuration.

FINANCING

A variety of methods and sources exist for financing the planning, design, construction, and operation of systems for the control of pollution from storm water runoff and combined sewer overflows. Financing sources may be categorized into two major types--capital construction, and operation and maintenance. In general, sources of funds that might be used wholly or partly by an agency for financing the design and construction of sewerage facilities include general obligation bonds, revenue bonds, special assessments, general tax revenues, Federal subsidies, private capital, state funds, and county funds. For operation and maintenance funding, ad valorem taxes, connection charges, participation fees, and service charges are commonly used methods for obtaining needed revenues.

The availability and application of these funds however is often dependent upon the exact nature and type of sewerage facilities, particularly with regard to Federal participation. To take full advantage of all potential funds therefore requires careful consideration of system objectives and features.

The Federal Water Pollution Control Act PL 84-660 as amended 33 USC 466 et seq. Clean Water Restoration Act of 1966, PL 89-753 is available for accelerating local wastewater treatment works construction. Grants can be made to any state, municipality, or other municipal or interstate agency for the construction of wastewater treatment and disposal systems, including collection of interceptor sewer flows. Several funding arrangements are available and include a 40-percent grant if the state also contributes at least 30 percent, a 50-percent grant if the state also contributes 25 percent and the system performs in conformance with enforceable water quality standards, and a 60-percent grant if the project conforms with a comprehensive metropolitan plan.

For those facilities not eligible for assistance under Federal Water Pollution Control Act, PL 84-660 as amended, direct grants are available to public agencies for financing up to 50 percent of the construction of storm water runoff and combined sewage collection systems under the Housing and Urban Development Act of 1965, PL 89-117, 79 STAT 490, 42 USC 3102 (Supp 1, 1965). The remaining 50 percent, when secured through general obligation or revenue bonds, can be financed by a loan received under the Public Works and Economic Development Act of 1965, PL 86-136, 79 STAT 552 that may run for as long as 40 years at an interest rate determined by Government borrowing costs. Should funding under HUD Act of 1965 be denied and the jurisdiction is designated as a redevelopment area, application under the Public Works and

Economic Development Act can be amended to request a 50 percent grant and a Federal loan for the remainder.

Separate storm water runoff sewers can be financed up to 100 percent under the Watershed Protection and Flood Prevention Act PL 83-566, PL 89-337, PL 89-4, Section 417, PL 87-703, STAT 608, 609 16 USC 1004, 1005 (Supp V), which provides technical and financial assistance to state and local agencies in planning, designing, and constructing watershed improvement works. If these facilities are located in redevelopment areas, they can also be funded up to 50 percent under Public Works and Economic Development Act of 1965, PL 89-156, STAT 552.

Additional sources of Federal subsidy are available for storage facilities when coupled with land conservation or open-space programs. The Housing Act of 1961, PL 87-70, 75 STAT 183, 42 USC 1500-1500e (1964) as amended, 42 USC 1500e (Supp 1, 1965) as amended, 42 USC 1500-1500d, provides for 50-percent matching grants to public agencies for acquiring, developing, and preserving open-space land. Grants may be increased up to 90 percent for projects that demonstrate improved methods of preserving urban open space. Financial assistance is available to states and their political subdivisions for all types of outdoor recreational areas and facilities, such as multi-purpose metropolitan parks, fishing grounds, and boating areas, through the Land and Water Conservation Fund Act of 1965, PL 88-578, 78 STAT 879, 16 USC 460D, 460L-11 (1964), 23 USC 120 (1964).

Due to the particular provisions and availability of all the foregoing funding sources, the most economical system overall may not necessarily be the one of least cost to the local community nor one that can be afforded locally because of smaller extramural funding assistance. Thus careful analysis must be made of alternative system costs and the communities ability or desire to locally finance it.

ENVIRONMENTAL FACTORS

Of growing concern to the populace is the condition of their surrounding environment and the modification that man has made or can effect. Therefore in attempts to improve and enhance his environs, he is aware of other effects, both deleterious and beneficial, that the construction and operation of other environment-improving facilities might have associated with them. An improvement in one condition can be undesirable if a degradation of another type occurs. Also, aesthetic values associated with major facilities should be considered and evaluated. Nuisances and inconvenience, however temporary, connected with specific alternatives must be taken into account. Although the environmental effects of alternative systems can be described, their absolute and relative importance requires a value judgment that is biased by one's vantage point.

Other factors requiring evaluation for alternative systems include benefits derived by the additional use of the facility for recreational purposes. An example might be the employment of a storm water runoff storage facility, filled wholly or partly with storm water runoff, for boating and picnicing.

For the alternative systems considered in this program, it was determined that, except in a cursory way, the differences that did exist in the alternatives with regard to their effect on the environment were small relative to other evaluation criteria and could be dismissed from further analysis. Much of each system is underground and except for the period of their placement, would present no irretrievable or perhaps even no measurable effects on the surrounding community. Recreational use of certain portions of alternative systems would be limited and hence of questionable value, due to the availability of adjacent waterways and other nearby recreational activities.

SYSTEMS EVALUATION

The expected effects on receiving water quality of storm water runoff and combined sewage discharges from a selected number of alternative systems that embrace in performance capability all systems considered in this program have been determined. Performance of the various systems was based on only discharges from the Combined Area to the Sacramento River at the downstream periphery of the Study Area, because the storm water runoff from the Separated Area was already represented in the Sacramento River quality used to establish background receiving water characteristics and because the pollution resulting from these storm water runoff discharges could be determined from evaluation of separated storm water runoff discharges from the Combined Area. Costs of alternative systems however represent total systems for the control of storm water runoff and combined sewage overflows derived from both the Combined Area and the Separated Area.

SYSTEM PERFORMANCE

Twelve systems for the treatment of combined sewage and storm water runoff, incorporating no treatment facilities (except chlorination) and combinations of different holding pond capacities and treatment processes, were simulated and the consequent receiving water quality distributions determined.

Rejection Criteria

On the basis of specified minimum receiving water quality criteria, it may be necessary to reject from further consideration several of the systems, thereby reducing the magnitude and complexity of the systems evaluation.

For the purposes of this program, values that are likely to be representative of requirements imposed on the Sacramento River at the planning horizon by the appropriate regulatory agency were selected for the three water quality criterion parameters--maximum value criterion, cumulative distribution criterion, and excession frequency criterion.

Values selected for the minimum BOD requirements were a maximum value of 11 mg/l, a cumulative distribution of greater than 6 mg/l less than 12 hr/yr, and an excession frequency of greater than 6 mg/l not more than twice each year. The maximum value criterion was selected on the basis that 11 mg/l represents roughly the dissolved oxygen saturation content of the Sacramento River which would be totally depleted of oxygen at BOD concentrations of 11 mg/l neglecting the reaction occurring downstream and the compensating undersaturation existing upstream from the point of wastewater discharge. Similarly, a BOD content of 6 mg/l could, if the receiving water upstream is of reasonably good quality, hence near oxygen saturation, reduce the dissolved oxygen levels to about 5 mg/l, which is usually satisfactory for the protection of a fisheries and wildlife resource. Concentrations of BOD in excess of 6 mg/l could further deplete the dissolved oxygen, but this condition is tolerable if it persists for only a short percentage of the total time, such as less than 12 hours out of each year, and if it seldom recurs, such as no more than two times a year on the average.

For the minimum total suspended solids requirements, values were set at 50 mg/l for the maximum value, greater than 10 mg/l less than 12 hr/yr for the cumulative distribution, and greater than 10 mg/l not more than twice a year for the excession frequency.

No minimum requirements were selected for the fecal coliform densities in the receiving water since it was assumed that all systems, including no treatment would provide chlorination facilities to adequately disinfect the wastewater discharges.

Combined Sewage

The cumulative distributions and excession frequencies of BOD concentrations occurring in the Sacramento River immediately after complete admixing of combined sewage and receiving water for six alternative systems are shown in Figure 53. Based on the selected performance criteria, Alternate A, which provides no holding pond or treatment process, fails all three tests of maximum value, cumulative distribution, and excession frequency. Alternates B and C, which are systems with no holding ponds and treatment processes capable of removing 30 and 60 percent of the total suspended solids, respectively, are unacceptable because they exceed both the maximum value criterion and the excession frequency criterion. Moreover, Alternate B, which consists of a 445 acre-ft holding basin upstream of a 60-percent removal process, does not meet the cumulative distribution criterion. Alternative D fails

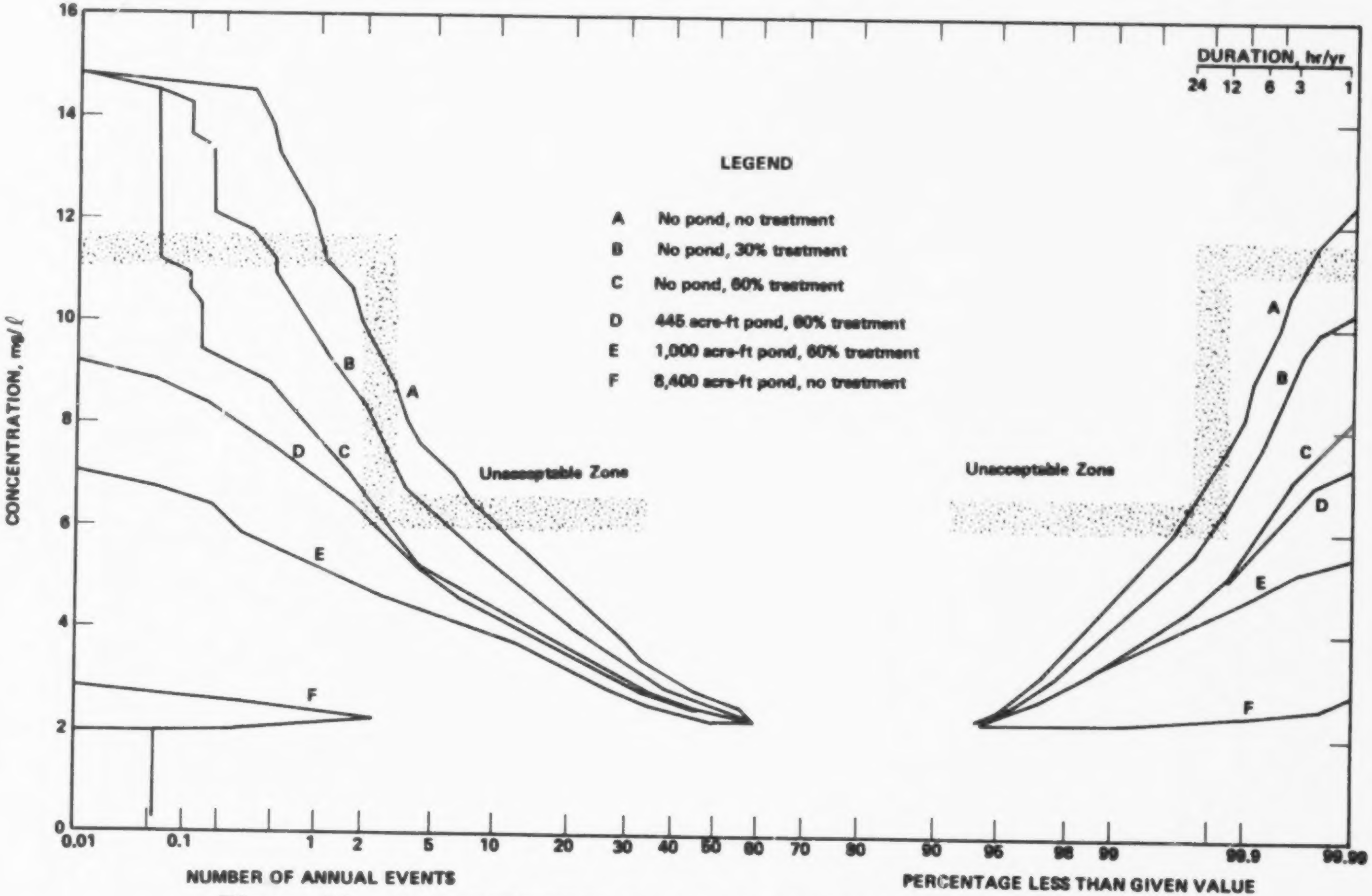

Figure 53. BOD CONTENT OF SACRAMENTO RIVER FOLLOWING COMBINED SEWAGE OVERFLOW DISCHARGE - YEAR 1992

only the excession frequency test, but thus is also an unacceptable system. Of the six alternatives simulated, only Alternatives E and F are acceptable for providing adequate BOD reductions in the combined sewage overflows. These systems contain, respectively, a 1,000 acre-ft holding pond followed with a process removing 60 percent of the total suspended solids, or 42 percent of the BOD, and an 8,400 acre-ft stabilization pond with no downstream treatment provided.

Figure 54 presents the incremental BOD concentration cumulative distributions and excession frequencies that would result in the Sacramento River from the discharge of the combined sewage overflow after various treatments. These distributions and expectancies are essentially the same as those including the natural background if the ordinate value is shifted 2 mg/l, which is the assumed constant background level in the River.

The results of the discharge to the Sacramento River of combined sewage overflows subjected to treatment by the six alternative systems with respect to suspended solids concentrations are shown in Figure 55. For all practical purposes, none of the alternative systems ranging from no treatment to extremely large holding ponds have any influence on the receiving water suspended solids concentrations due to the relatively small flows of combined sewage that contain for the most part suspended solids concentrations no greater and in some cases less than that of the River.

Therefore to assess the relative performance of the various alternative systems, cumulative distributions and excession frequencies of suspended solids concentrations were established for discharge of the treated combined sewage overflows to the Sacramento River if it were totally devoid of suspended solids. These results, presented in Figure 56, reveal that of the six systems considered, only Alternatives A and B, which provide for no ponds and either no or 30-percent removal of suspended solids, are unacceptable since they exceed the excession frequency criterion of 10 mg/l occurring not more than twice each year.

Although the fecal coliform densities resulting from discharge of the combined sewage into the Sacramento River are not considered in the evaluation of systems performance as explained earlier, the density cumulative distributions and excession frequencies were established for discharge from four alternative systems consisting only of retention ponds ranging in size from 0 to 8,400 acre-ft. Figure 57 indicates that none of these treatment systems has a measurable effect on natural Sacramento River fecal coliform concentrations. From Figure 58, which presents the relative performance of the various alternative systems (containing no disinfection facilities), it is seen that only the use of an exceptionally large stabilization pond will produce substantial reductions in fecal coliform populations.

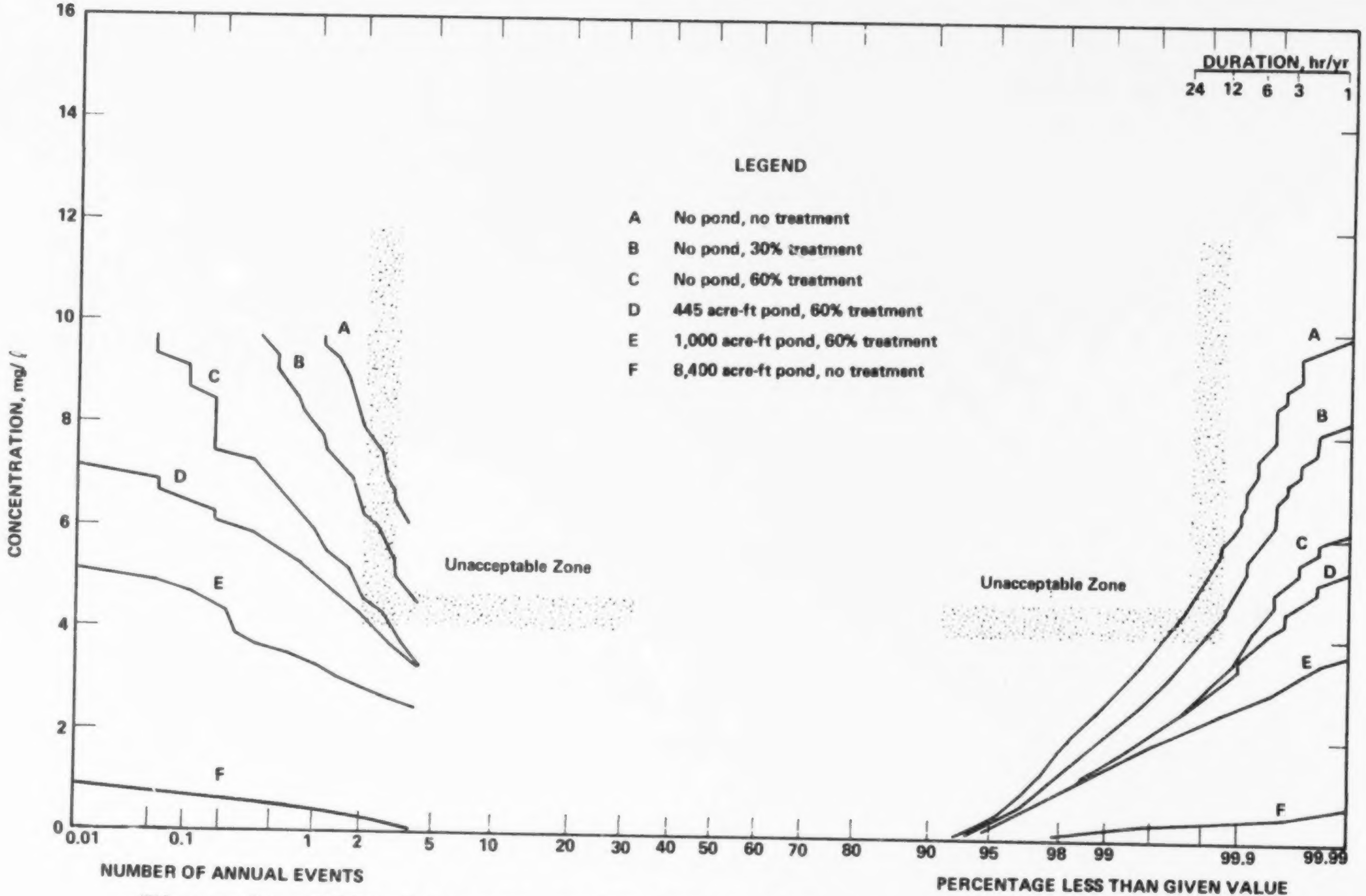

Figure 54. INCREMENTAL ADDITION OF BOD TO SACRAMENTO RIVER BY COMBINED SEWAGE OVERFLOW DISCHARGE - YEAR 1992

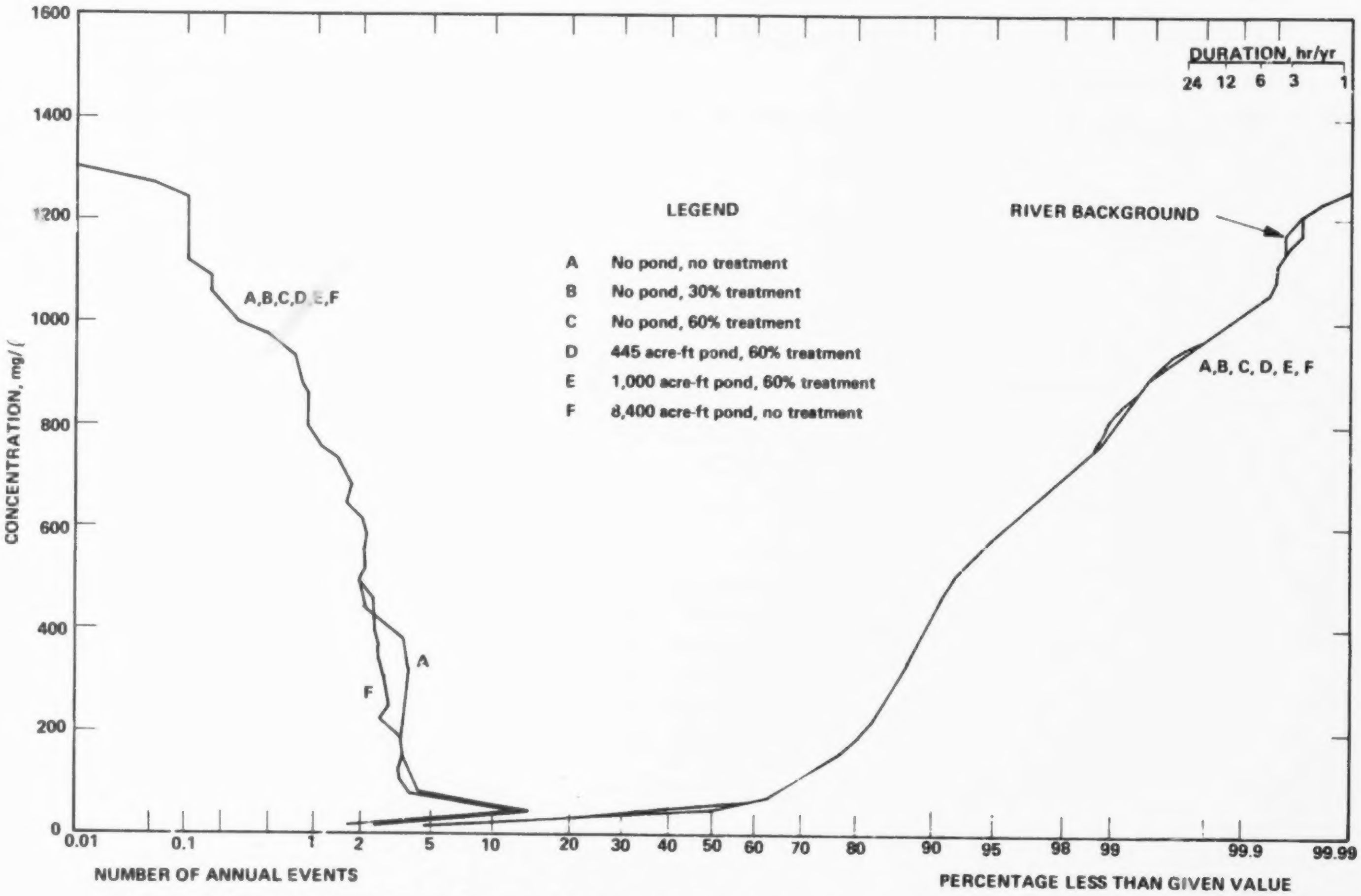

Figure 55. SUSPENDED SOLIDS CONTENT OF SACRAMENTO RIVER FOLLOWING COMBINED SEWAGE OVERFLOW DISCHARGE - YEAR 1992

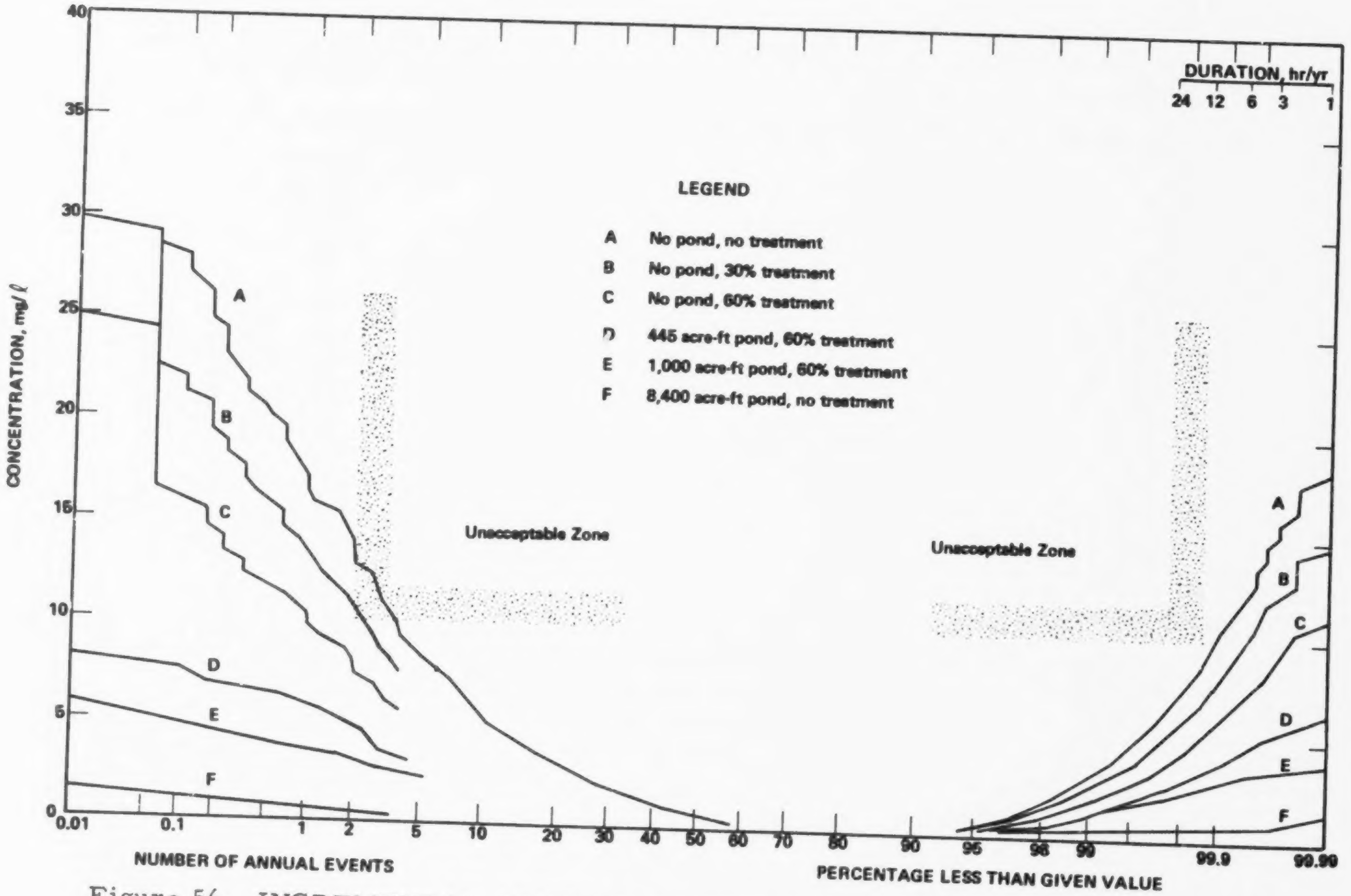

Figure 56. INCREMENTAL ADDITION OF SUSPENDED SOLIDS TO SACRAMENTO RIVER BY COMBINED SEWAGE OVERFLOW DISCHARGE - YEAR 1992

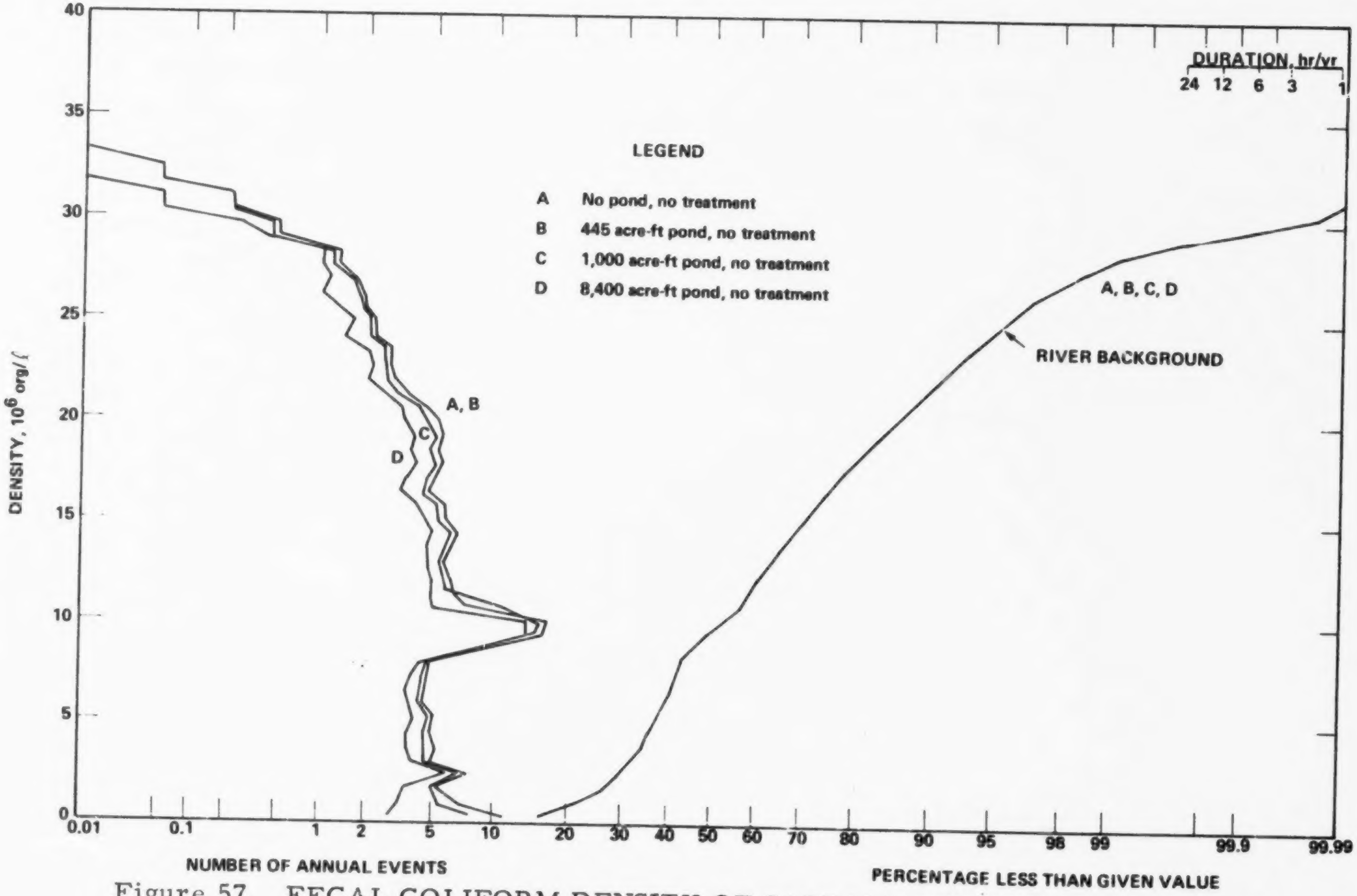

Figure 57. FECAL COLIFORM DENSITY OF SACRAMENTO RIVER FOLLOWING COMBINED SEWAGE OVERFLOW DISCHARGE - YEAR 1992

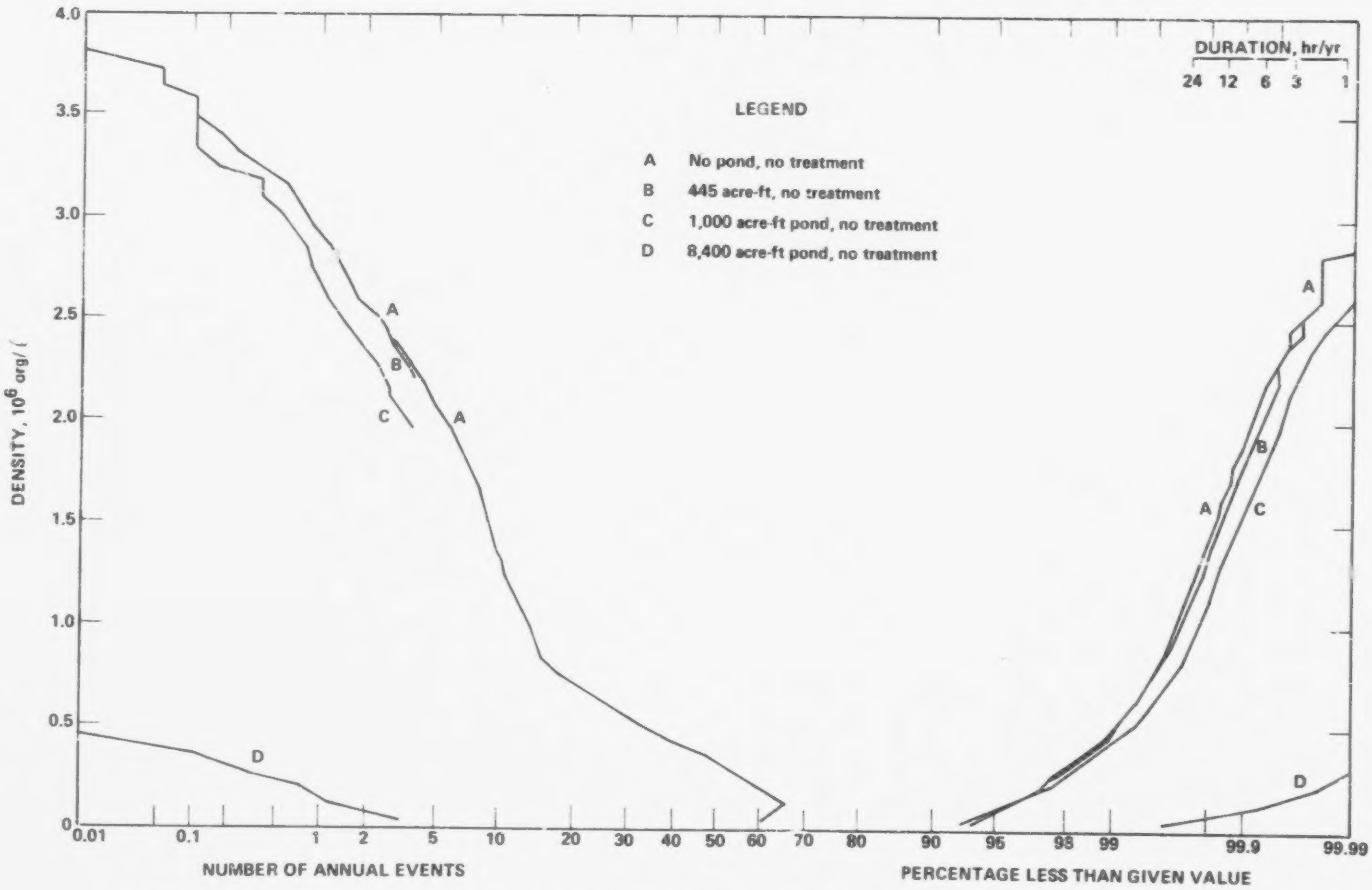

Figure 58. INCREMENTAL ADDITION OF FECAL COLIFORMS TO SACRAMENTO RIVER BY COMBINED SEWAGE OVERFLOW DISCHARGE - YEAR 1992

Storm Water Runoff

The effects on the BOD content of Sacramento River of the storm water discharges from the Combined Area that are subjected to varying degrees of treatment are shown in Figure 59. Only Alternatives E and F, consisting of a 625 acre-ft retention pond with a facility to remove 30 percent of the suspended solids and a 3,700 acre-ft stabilization pond with no further treatment, respectively, are acceptable systems according to the selected performance criteria for BOD. Whereas all six alternative systems considered have acceptable BOD concentration cumulative distributions, three alternatives are rejected on the basis of both excession frequency and maximum value while a fourth alternative is unacceptable as a result only of failing the maximum value criterion.

Distributions of the incremental addition of BOD to the receiving waters from the six alternative systems are presented in Figure 60.

The cumulative distributions and excession frequencies of suspended solids in the Sacramento River subsequent to storm water discharge is shown in Figure 61. No differences in performance of the six alternative systems for removal of suspended solids are noted upon discharge of storm water runoff to the Sacramento River, due to the high indigenous suspended solids concentration and relative flow of receiving waters. The relative performance of the various systems in reducing suspended solids content of the receiving waters, however, are presented in Figure 62. Only Alternate A, which consists of no system, is unacceptable, and then only because of its excessive excession frequency.

Fecal coliform densities that would occur in the Sacramento River as a result of the discharge of storm water runoff that is not disinfected by chlorination but is subjected to retention in ponds ranging in size from 0 to 3,700 acre-ft are shown in Figure 63. No measurable differences are noted because the flows and bacterial contents of the storm water runoff are markedly lower than those that are extant in the receiving waters. The relative performance of the several systems considered in reducing fecal coliform populations are depicted in Figure 64.

SYSTEMS COST EFFECTIVENESS

The complete system for the abatement of combined sewer overflows and storm water runoff in the Study Area must include facilities for the treatment of the separated storm water from the Separated Area and treatment of either the combined sewage or separated storm water runoff from the Combined Area. Separated sanitary sewage or dry-weather flow in combined sewers in the Combined Area would be processed in the existing Main Treatment Plant and thus are excluded from the systems under consideration. Two methods of handling the storm water runoff from the Separated Area were explored--discharge directly to

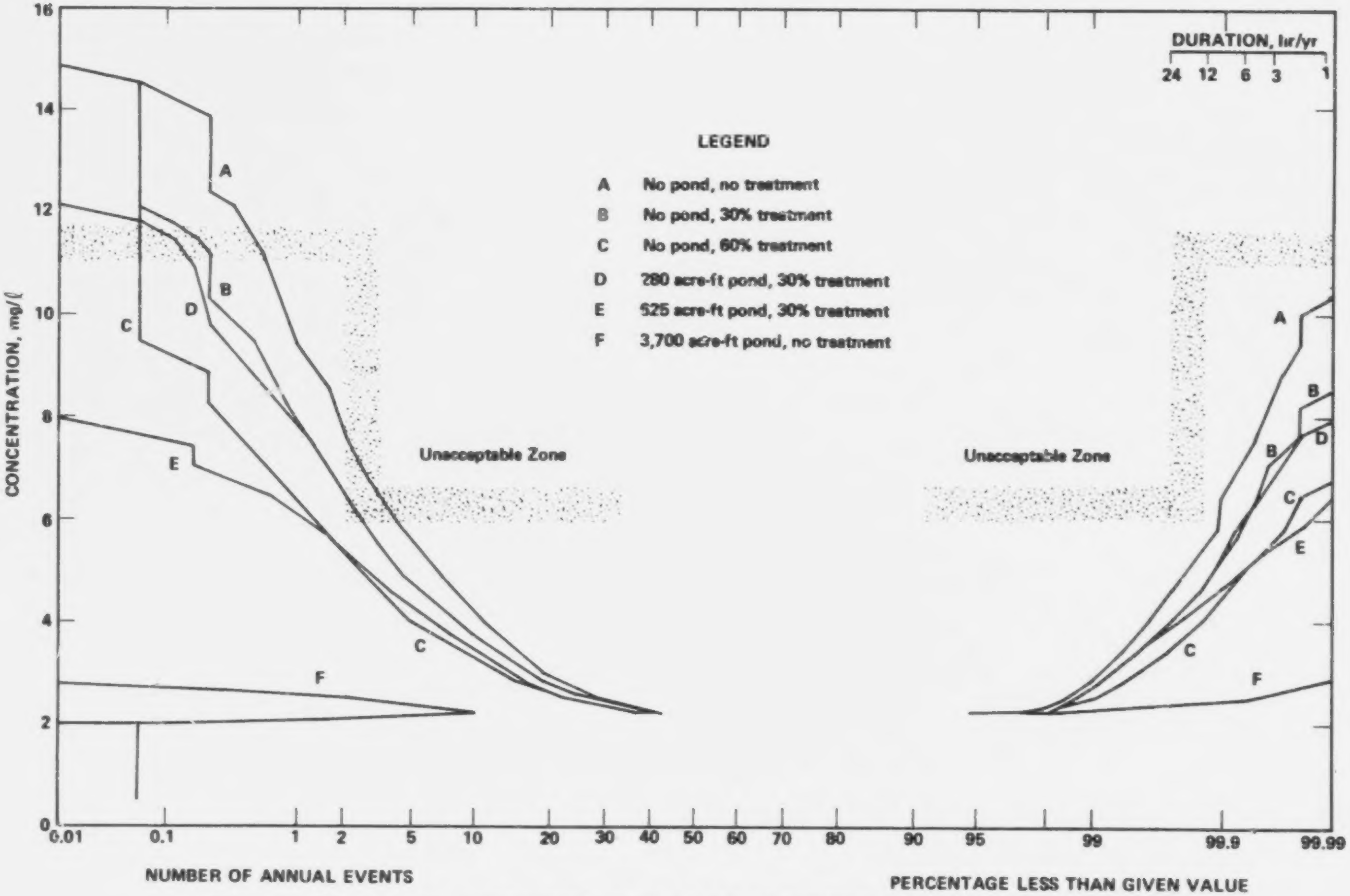

Figure 59. BOD CONTENT OF SACRAMENTO RIVER FOLLOWING STORM WATER RUNOFF DISCHARGE - YEAR 1992

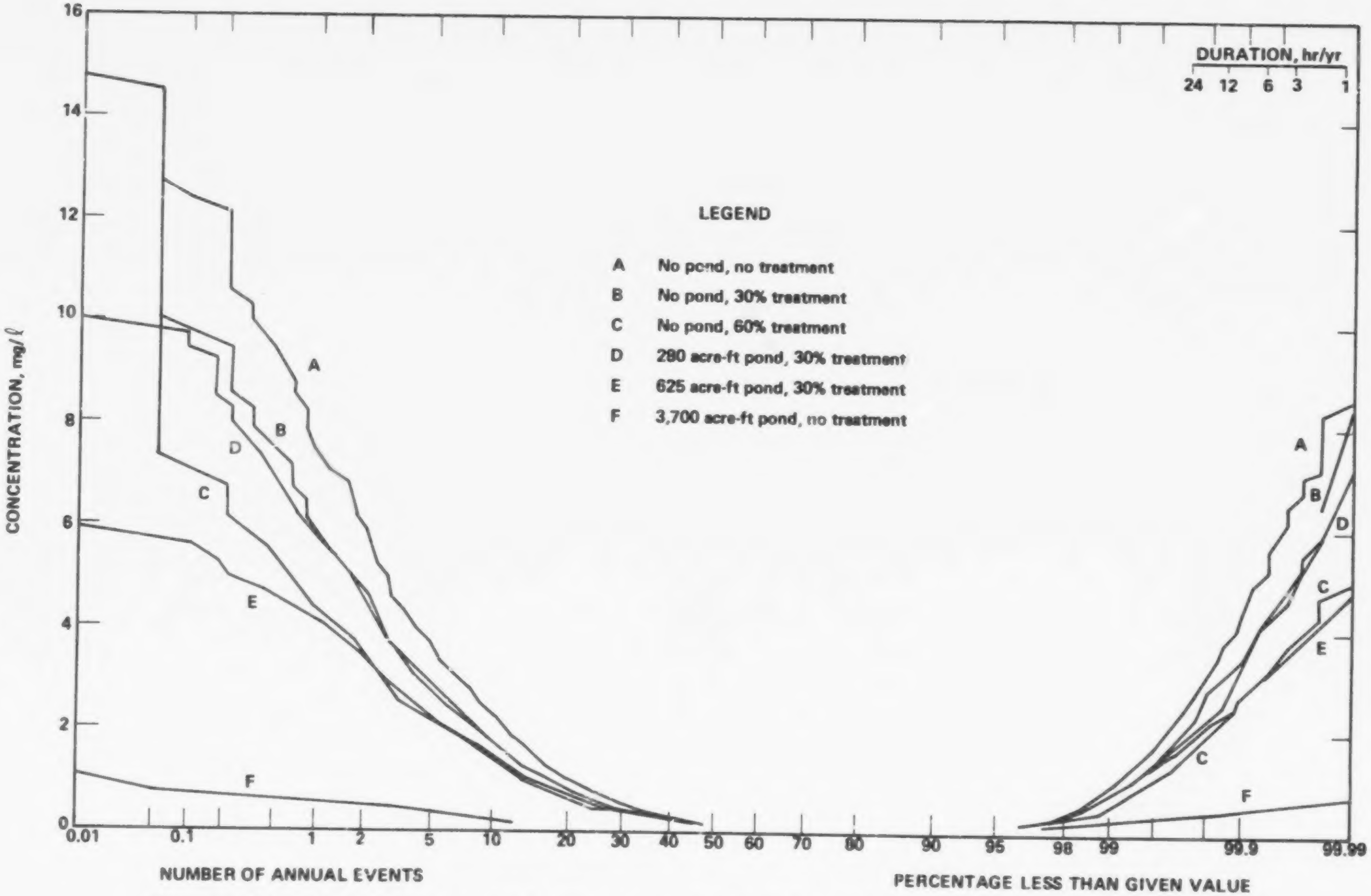

Figure 60. INCREMENTAL ADDITION OF BOD TO SACRAMENTO RIVER BY STORM WATER RUNOFF DISCHARGE - YEAR 1992

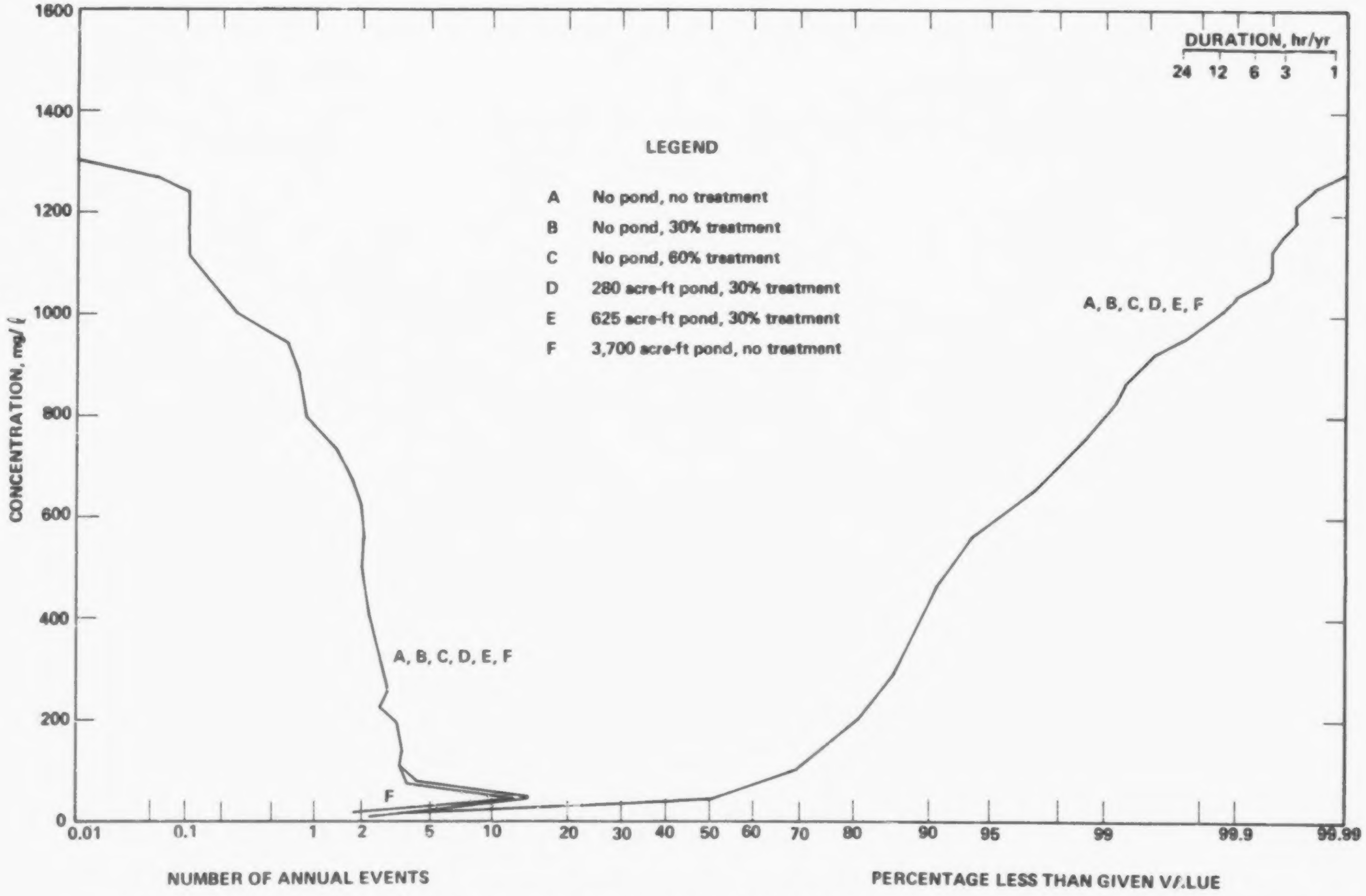

Figure 61. SUSPENDED SOLIDS CONTENT OF SACRAMENTO RIVER FOLLOWING STORM WATER RUNOFF DISCHARGE - YEAR 1992

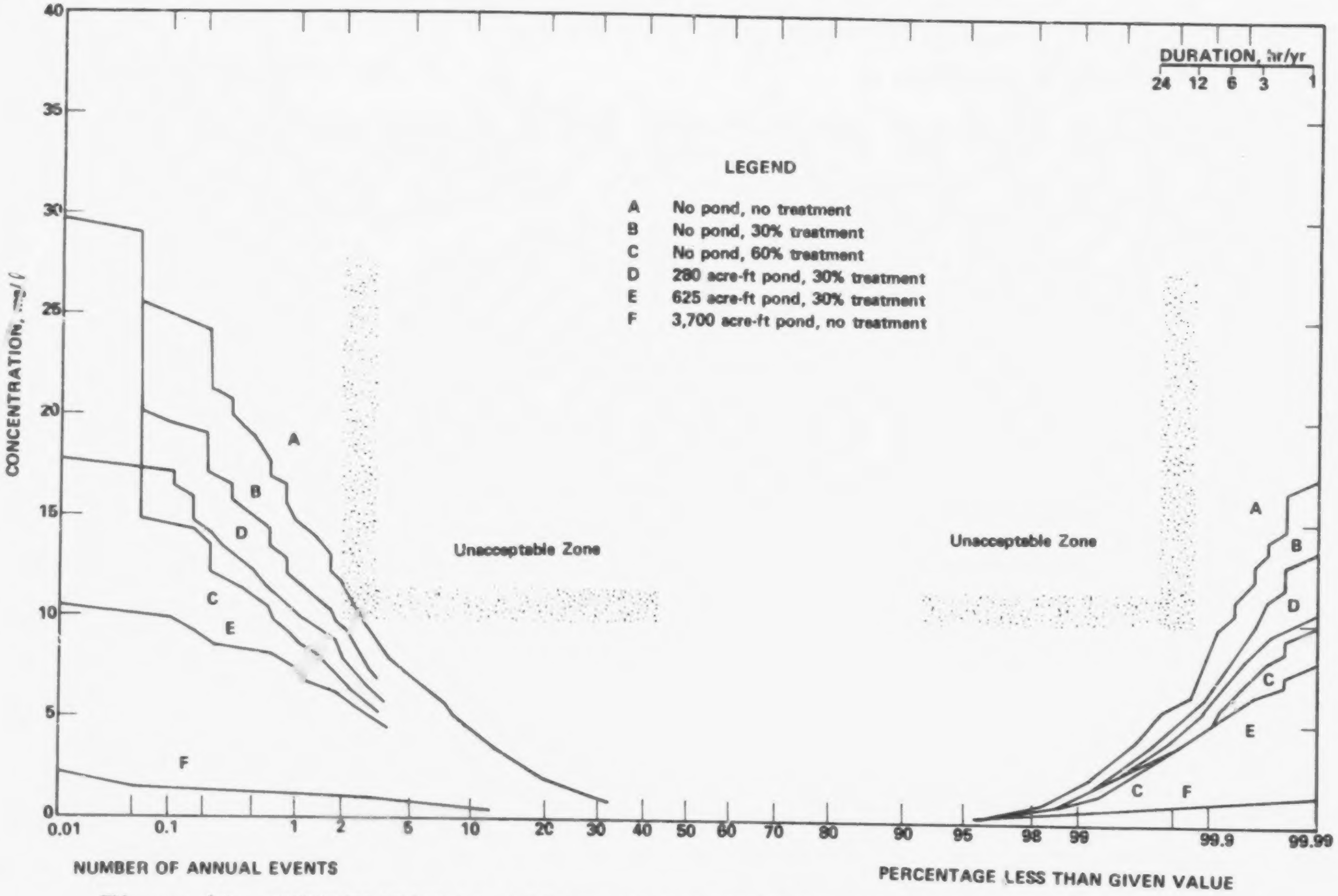

Figure 62. INCREMENTAL ADDITION OF SUSPENDED SOLIDS TO SACRAMENTO RIVER BY STORM WATER RUNOFF DISCHARGE - YEAR 1992

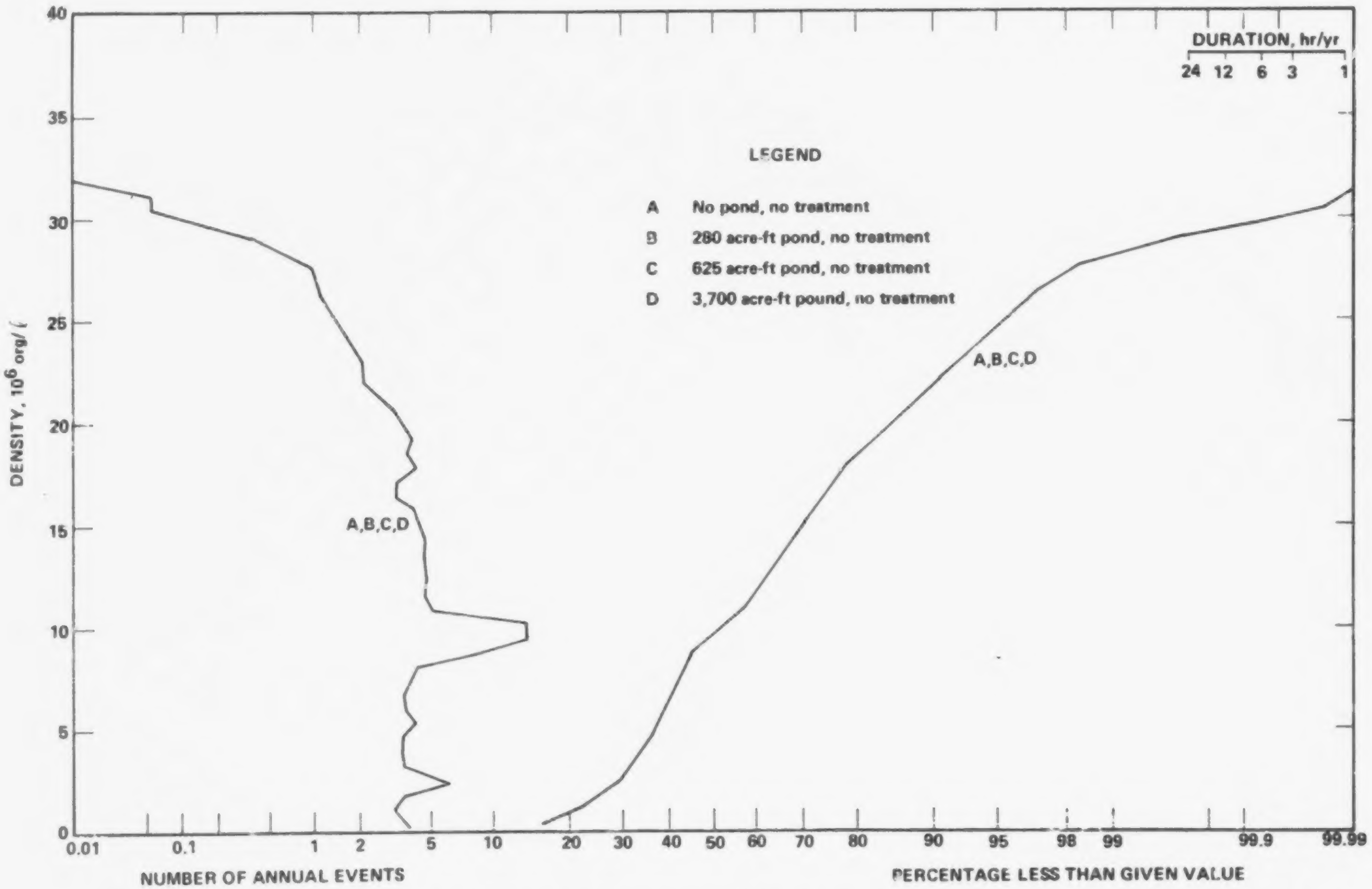

Figure 63. FECAL COLIFORM DENSITY OF SACRAMENTO RIVER FOLLOWING STORM WATER RUNOFF DISCHARGE - YEAR 1992

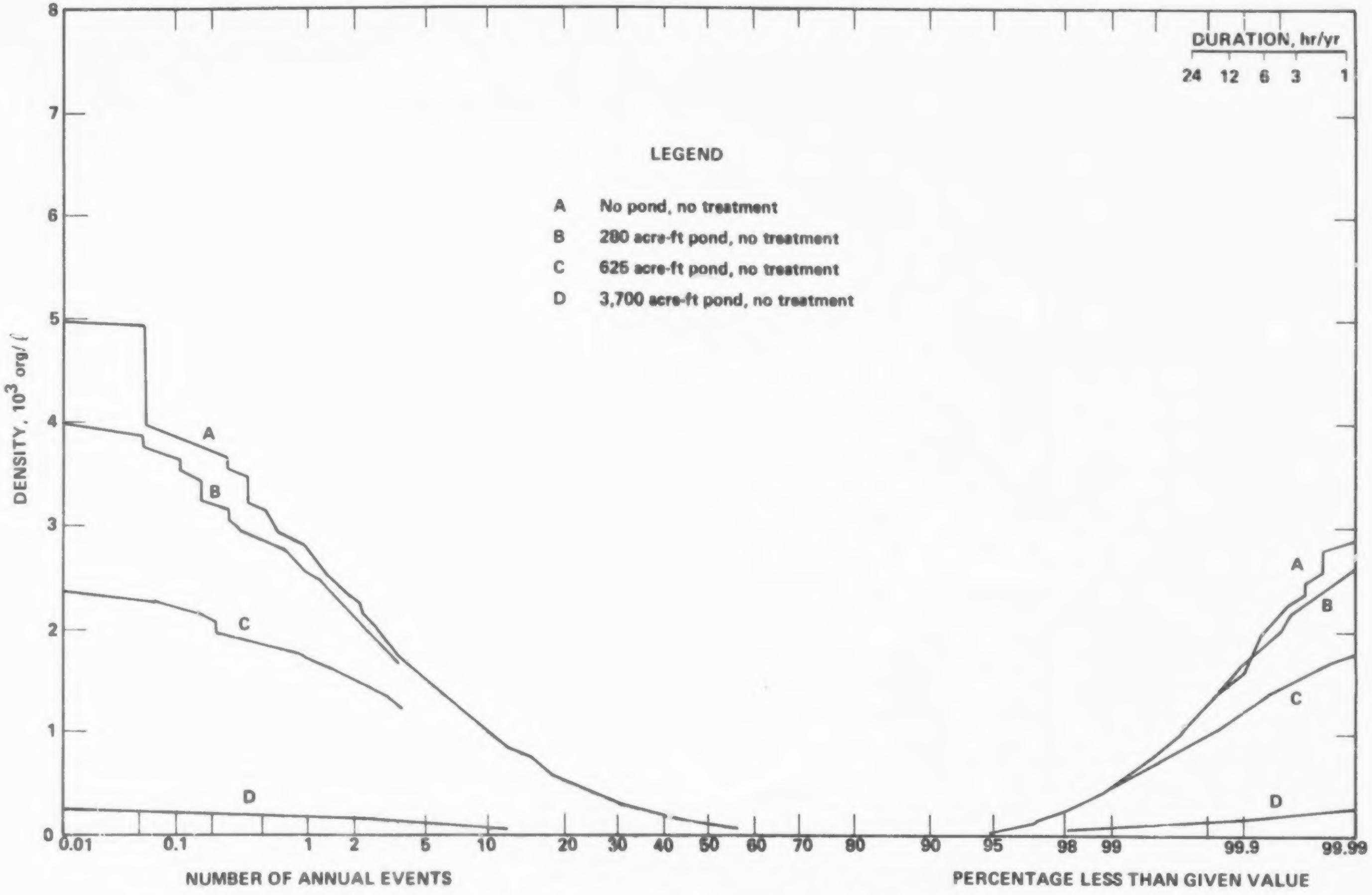

Figure 64. INCREMENTAL ADDITION OF FECAL COLIFORMS TO SACRAMENTO RIVER BY STORM WATER RUNOFF DISCHARGE - YEAR 1992

the receiving waters from either individual or collective facilities and discharge to the Combined Area system prior to treatment of all wastewaters from the Study Area in a single facility.

Applying to the Separated Area and Total Area the results of the performance simulation for the discharge of treated storm water runoff and combined sewer overflows to the Sacramento River from the Combined Area, a number of acceptably performing systems are available for evaluation. The general features and the total annual cost of these systems designed for a 5-year storm recurrence interval are presented in Table 31.

If the conveyance system remains combined, the minimum annual cost of $6.94 million is provided by the system comprised of separate collection and treatment in the Separated Area and in the Combined Area. For the Separated Area, the system consists of an augmented storm water conveyance network, a 275 acre-ft surface holding pond, an air flotation unit capable of 30-percent suspended solids reduction, and a chlorination facility. The separated storm water runoff treatment facility for the Separated Area is located at the American River Park. In the Combined Area, the existing combined sewer network is augmented to deliver wet-weather combined sewage to the Sacramento Port location across the Sacramento River, where treatment and disposal facilities consist of an 890 acre-ft holding basin, an air flotation unit capable of removing 60 percent of the suspended solids, and a chlorination facility.

The minimum annual cost associated with a system that provides for complete separation of sanitary sewage and storm water runoff is $7.04 million, and it too consists of separate treatment facilities in the Separated Area and the Combined Area. This system is the same in the Separated Area as previously described for the system employing combined sewers and consists in the Combined Area of a new storm water conveyance network, a 570 acre-ft surface reservoir, an air flotation facility to remove 30 percent of the suspended solids, and a chlorination station with the treatment facilities located across the River at the Sacramento Port location.

It should be noted that a substantial portion of the total annual costs for the acceptably performing systems utilizing combined sewers result from the enlargement or augmentation of the existing sewers to provide adequate hydraulic capacity in the conveyance lines. For the system utilizing combined sewers, for example, augmentation of the existing conveyance system would cost $3.10 million, or nearly one half of the total system cost. The system embodying complete separation of sanitary and storm water flows requires, however, an annual expenditure of $0.52 million for enlargement of the existing sewers, whereas the remaining portion of the $3.68 million for total conveyance provisions is necessary for new separated storm water sewer construction, operation, and maintenance in the Combined Area.

Table 31

ACCEPTABLY PERFORMING SYSTEMS FOR CONTROL OF STORM WATER RUNOFF AND COMBINED SEWER OVERFLOWS FROM STUDY AREA

Sewer Type	Service Area	Facilities Description*	Annual Cost, million dollars	
			Sub-system	System
Combined	Combined	890 acre-ft pond, 60% air flotation, Sacramento Port	4.84	
		950 acre-ft pond, 60% air flotation, South Sacramento	6.27	
		2,600 acre-ft pond, sewage treatment plant, Sacramento Port	5.78	
		2,600 acre-ft pond, sewage treatment plant, South Sacramento	6.96	
		8,400 acre-ft stabilization pond, Sacramento Port	6.01	
		8,400 acre-ft stabilization pond, South Sacramento	7.33	
	Separated	280 acre-ft pond, 30% air flotation, American River Park	2.10	
		420 acre-ft pond, 60% air flotation, American River Park	2.21	
		810 acre-ft stabilization pond, American River Park	2.23	
	Study	1,500 acre-ft pond, 60% air flotation, Sacramento Port		7.48
		1,900 acre-ft pond, 60% air flotation, South Sacramento		9.27
		4,800 acre-ft pond, sewage treatment plant, Sacramento Port		7.21
		4,800 acre-ft pond, sewage treatment plant, South Sacramento		8.74
Separated	Combined	570 acre-ft pond, 30% air flotation, Sacramento Port	4.94	
		770 acre-ft pond, 30% air flotation, South Sacramento	6.37	
		640 acre-ft pond, 30% mechanical screens, Sacramento Port	5.09	
		940 acre-ft pond, 30% mechanical screens, South Sacramento	6.46	
		940 acre-ft pond, 60% air flotation, Sacramento Port	5.22	
		1,200 acre-ft pond, 60% air flotation, South Sacramento	6.58	
		1,400 acre-ft pond, sewage treatment plant, Sacramento Port	5.65	
		1,400 acre-ft pond, sewage treatment plant, South Sacramento	6.83	
		3,700 acre-ft stabilization pond, Sacramento Port	5.29	
		3,700 acre-ft stabilization pond, South Sacramento	6.60	
	Separated	280 acre-ft pond, 30% air flotation, American River Park	2.10	
		420 acre-ft pond, 60% air flotation, American River Park	2.21	
		810 acre-ft stabilization pond, American River Park	2.23	
	Study	800 acre-ft pond, 30% air flotation, Sacramento Port		7.32
		1,000 acre-ft pond, 30% air flotation, South Sacramento		8.93
		900 acre-ft pond, 30% mechanical screens, Sacramento Port		7.51
		1,000 acre-ft pond, 30% mechanical screens, South Sacramento		9.17
		1,000 acre-ft pond, 60% air flotation, Sacramento Port		7.71
		1,200 acre-ft pond, 60% air flotation, South Sacramento		9.35
		2,300 acre-ft pond, sewage treatment plant, Sacramento Port		6.32
		2,300 acre-ft pond, sewage treatment plant, South Sacramento		7.68

*All ponds are surface ponds.

Not only does the combined system provide a less costly system than the totally separated system but it also produces less overall degradation of the receiving waters, albeit small. The water quality of the Sacramento River resulting from the discharge of treated wet weather combined sewage from the Combined Area, which represents the major portion of the total wastewater discharge from the Study Area, by the least costly water pollution abatement system is shown in Table 32, which also presents similar data for the totally separated system.

Table 32

COMPARISON OF RECEIVING WATER QUALITY CHARACTERISTICS FROM THE LEAST COSTLY ACCEPTABLY PERFORMING COMBINED AND SEPARATED SEWER SYSTEMS
Year 1992

Receiving Water Quality Parameter	Combined Sewer System		Separated Sewer System	
	BOD, mg/l	Suspended Solids,* mg/l	BOD, mg/l	Suspended Solids,* mg/l
Average Value	2.15	0.045	2.12	0.046
Maximum Value	7.05	5.70	7.95	10.5
Value exceeded less than 12 hr/yr	4.5	2.6	4.4	4.3
Value exceeded only twice annually	4.8	3.1	5.4	6.1

*Incremental addition

FINANCING OPPORTUNITIES

The availability of financing sources and methods differs for the construction of the combined sewer system and the separated sewer system, as shown in Table 33. The values listed in Table 33 indicate the maximum funding eligibility under existing laws, and not necessarily the amounts that would actually become available. Also, the actual funds approved by the Federal Government or State determine the amount of local matching funds required, and amy influence the choice of local funding methods.

Table 33

MAXIMUM FUNDING ELIGIBILITIES FOR COMBINED SEWAGE AND STORM WATER RUNOFF WATER POLLUTION ABATEMENT SYSTEMS

System	Funding Source and Method*	Funding Eligibility, %			Construction Cost Contribution, million dollars		
		Federal	State	Local	Federal	State	Local
Combined Sewer							
Conveyance Subsystem							
Augment Storm Sewers	1	100			8.02		
Augment Combined Sewers	2	50			21.88		
	3			50			21.88
Storage Subsystem							
Separated Storm Water	4	100			17.37		
Combined Sewage	5	60			5.73		
	6		25			2.39	
	3			15			1.43
Treatment Subsystem							
Separated Storm Water	7	100			1.24		
Combined Sewage	5	60			3.00		
	6		25			1.25	
	3			15			0.75
Disposal Subsystem							
Separated Storm Water	7	100			0.30		
Combined Sewage	5	60			0.38		
	6		25			0.16	
	3			15			0.10
Totals					57.92	3.80	24.16
Separated Sewer							
Conveyance Subsystem							
New Storm Sewers	1	100			53.31		
Augment Storm Sewers	1	100			8.02		
Augment Sanitary Sewers	2	50			0.24		
	3			50			0.24
Storage Subsystem							
Separated Area	4	100			17.37		
Combined Area	4	100			8.75		
Treatment Subsystem							
Separated Area	7	100			1.24		
Combined Area	7	100			3.01		
Disposal Subsystem							
Separated Area	7	100			0.30		
Combined Area	7	100			0.67		
Totals					92.91	0.00	0.24

* 1 WP&FP Act PL83-566, PL89-337, PL89-4, PL87-703
2 HUD Act 1965 PL89-117
3 General Obligation and/or Revenue Bonds
4 WP&FP Act PL83-566
5 FWPC Act PL84-660
6 Senate Bill 647
7 WP&FP Act PL83-566, PL89-337, PL89-4

SYSTEM SELECTION

The final selection of the system, which within the constraints best satisfies all requirements imposed upon the performance, cost, acceptance, implementation, operation, and maintenance of facilities for the abatement of water pollution from storm water runoff and combined sewer overflows in the Sacramento area, can be performed only after a much greater local involvement and commitment than was required and provided in this program. On the basis of the information and evaluations developed and presented herein, however, a system can be recommended for selection by the City of Sacramento.

Although the least costly, acceptably performing system is characterized by the retention and augmentation of existing combined sewers in the Combined Area, a system which provides for the complete separation of sanitary sewage and storm water runoff is only slightly more expensive but presents a much greater opportunity for extramural funding support from the Federal Government. Under maximum subsidy, the financial commitment by the local community would be only about 0.25 percent, or $240,000, of the total construction cost of $93.15 million for this system. Minimum local financial participation in the less costly combined sewer system would be about 28 percent of the total construction cost of $85.88 million, or $24.16 million. On an annualized basis, the separated sewer system is only 1.4 percent more costly than the combined sewer system.

The recommended system, therefore, that will protect the receiving waters from the discharge of storm water runoff and combined sewer overflows from the Study Area involves the complete separation of sanitary sewage and storm flows and provides for separate conveyance and treatment systems in the Combined Area and the Separated Area. The Combined Area system would be comprised of a new gravity sewer system for the collection and conveyance of storm water runoff from the Combined Area to a treatment facility at the Sacramento Port location. This treatment facility would consist of a 570 acre-ft surface reservoir, a dissolved air flotation unit capable of reducing total suspended solids by 30 percent and total BOD by 21 percent, and a chlorination station.

Sanitary sewage from the Study Area would be collected and conveyed to the existing sewage treatment plant by an augmented existing combined sewer system.

In the Separated Area, the system would consist of an augmented existing storm water runoff collection network and a new interceptor sewer for conveyance of the storm water runoff to a common treatment facility located at American River Park. Treatment facilities there would consist of a 280 acre-ft surface holding pond, a dissolved air flotation unit for removal of 30 percent of the total suspended solids and 21 percent of the total BOD, and a chlorination station.

Section IX

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Section X

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Section XI

APPENDICES

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Appendix A

DRY-WEATHER COMBINED FLOW CHARACTERISTICS

Station Number	Date	Time	Station Flow Depth, ft	Pipe Diameter, in.	Flow, cfs	Total Suspended Solids, mg/l	Volatile Suspended Solids, mg/l	Settleable Solids, mg/l	BOD, mg/l	COD, mg/l	pH	Fecal Coliforms, org/l
Run No. 1												
1	14 May 69	0912	4.95	60	68.6	46	3	6	57	196	7.9	1.6 x 10^7
2		0905	1.28	114	11.6	66	49	23	80	137	7.4	4.2 x 10^6
3		0900	3.58	108	98.0	314	200	270	113	200	7.4	2.1 x 10^7
4		0958	1.60	60	16.5	86	59	86	180	328	7.5	2.0 x 10^7
5		0950	2.05	60	2.30	178	41	18	209	279	7.8	1.8 x 10^7
6		1008	0.10	10	0.19	101	81	30	250	186	7.5	1.1 x 10^7
7		0945	0.25	56	0.2	275	122	263	195	510	8.2	2.9 x 10^7
8		0940	2.02	108	58.8	148	108	98	168	225	7.7	4.0 x 10^6
9		0940	1.80	84	41.8	137	46	77	27	10	7.7	1.4 x 10^6
10		0955	0.82	42	4.2	88	50	41	132	200	7.3	9.0 x 10^6
11		1005	0.70	30	1.4	20.5	2.0	0	53	69	7.5	2.2 x 10^6
12		1015	1.35	45	8.0	370	301	50	83	510	7.4	6.0 x 10^6
13		0925	3.4	60	69.6	24	18	20	121	127	7.6	1.4 x 10^7
14		0900	0.35	24	0.3	206	154	133	176	426	7.4	2.0 x 10^7
15		0850	0.36	45	0.4	260	72	233	188	456	7.8	2.9 x 10^7
16		0910	0.75	24	4.9	167	147	167	25	622	7.6	7.5 x 10^4
Run No. 2												
1	14 May 69	1106	6.80	60	66.2	30	21	21	56	157	7.5	1.8 x 10^7
2		1135	2.83	114	57.4	50	45	56	124	211	7.5	4.0 x 10^7
3		1131	6.08	108	238.0	190	187	176	153	132	7.5	3.0 x 10^7
4		1217	2.65	60	41.0	52	32	42	110	235	7.6	9.0 x 10^6
5		1203	2.85	60	40.4	50	6	32	160	216	7.7	2.7 x 10^6
6		1212	0.15	10	0.28	75	58	53	150	294	7.6	7.0 x 10^6
7		1130	0.25	56	0.2	109	56	87	128	216	7.7	1.1 x 10^7
8		1125	2.07	108	61.5	192	119	172	180	446	7.4	6.0 x 10^6
9		1149	2.10	84	56.5	58	12	51	15	29	7.9	2.0 x 10^5
10		1135	0.77	42	3.0	111	39	80	250	216	7.2	2.0 x 10^7
11		1145	0.75	30	1.8	25	14	25	43	78	7.8	1.8 x 10^6
12		1150	1.40	45	8.4	424	348	393	180	661	7.3	7.0 x 10^6
13		1120	5.25	60	70.0	32	12	30	64	118	7.5	1.6 x 10^7
14		1105	0.40	24	0.4	143	92	142	260	333	7.9	8.0 x 10^6
15		1055	0.41	45	0.6	177	100	167	98	421	7.5	2.0 x 10^8
16		1110	0.85	24	6.1	50	0	2	200	573	7.2	2.0 x 10^6
Run No. 3												
1	14 May 69	1301	8.10	60	72.8	3,338	2,536	3,066	82	1,648	6.6	1.6 x 10^7
2		1333	3.68	114	90.0	137	0	135	167	135	7.7	2.9 x 10^7
3		1329	6.88	108	278.0	190	167	180	160	200	7.0	1.4 x 10^7
4		1457	3.20	60	55.0	72	51	63	154	152	9.0	1.2 x 10^7
5		1351	3.60	60	56.7	177	114	171	139	200	8.0	1.0 x 10^7
6		1407	0.23	10	0.5	194	177	76	88	431	7.6	7.0 x 10^6
7		1330	0.25	56	0.2	202	119	179	177	274	7.3	1.5 x 10^7
8		1325	2.57	108	94.5	198	146	186	300	426	7.6	2.3 x 10^7
9		1344	3.05	84	113.7	65	25	65	45	5	7.6	3.0 x 10^6
10		1335	0.72	42	2.6	57	47	57	150	78	7.6	2.5 x 10^7
11		1345	0.80	30	2.1	54	34	17	76	152	7.7	7.0 x 10^6
12		1350	1.35	45	8.0	248	176	219	425	730	7.5	1.6 x 10^7
13		1314	6.30	60	79.5	72	19	71	81	93	7.8	1.7 x 10^7
14		1305	0.30	24	0.2	72	47	67	174	230	7.5	1.5 x 10^7
15		1255	0.36	45	0.4	56	48	6	185	200	7.6	1.4 x 10^7
16		1315	0.85	24	6.1	407	338	390	800	588	7.5	2.1 x 10^7

Appendix A (continued)

DRY-WEATHER COMBINED FLOW CHARACTERISTICS

Station Number	Date	Time	Station Flow Depth, ft	Pipe Diameter, in.	Flow, cfs	Total Suspended Solids, mg/l	Volatile Suspended Solids, mg/l	Settleable Solids, mg/l	BOD, mg/l	COD, mg/l	pH	Fecal Coliforms, org/l
Run No. 4												
1	14 May 69	1458	8.20	60	68.4	9,274	0	9,065	186	2,704	6.8	1.6×10^8
2		1523	3.78	114	99.0	22	14	22	153	235	7.5	2.3×10^7
3		1519	7.03	108	285.0	150	133	135	178	299	7.5	2.4×10^7
4		1551	3.40	60	59.7	8	0	6	113	157	7.3	1.6×10^7
5		1547	3.81	60	60.5	35	12	3	203	191	7.2	1.4×10^7
6		1535	0.20	10	0.42	31	8	31	205	382	7.7	1.3×10^7
7		1525	0.40	56	0.4	136	109	129	280	309	7.4	1.6×10^7
8		1520	2.62	108	97.8	420	378	307	550	578	7.3	9.0×10^6
9		1535	3.20	84	124.0	55	32	17	34	34	7.5	1.6×10^6
10		1530	0.72	42	2.6	28	9	5	216	83	7.4	2.0×10^7
11		1545	0.80	30	2.1	44	28	30	74	74	7.8	3.0×10^6
12		1550	1.30	45	7.2	271	230	235	375	696	7.4	4.0×10^6
13		1513	6.50	60	64.4	322	309	315	68	88	7.5	1.8×10^7
14		1500	0.40	24	0.4	89	80	43	175	274	7.3	1.4×10^7
15		1450	0.36	45	0.4	140	105	41	300	358	7.5	5.0×10^6
16		1510	0.90	24	6.8	365	239	327	138	583	7.4	6.8×10^6
Run No. 5												
1	14 May 69	1700	8.15	60	66.2	142	119	81	38	201	7.4	1.4×10^7
2		1726	3.73	114	97.0	631	185	617	185	382	7.2	2.8×10^7
3		1721	6.90	108	278.0	51	0	17	189	1,132	7.5	1.3×10^7
4		1754	3.35	60	58.8	124	118	107	54	397	7.5	7.0×10^7
5		1751	3.75	60	59.5	107	107	48	170	456	7.7	1.0×10^7
6		1740	0.20	10	0.42	60	60	21	209	764	9.1	1.5×10^7
7		1725	0.35	56	0.3	83	63	83	190	255	7.4	6.0×10^6
8		1720	2.57	108	74.5	122	97	122	128	417	7.5	1.1×10^7
9		1740	2.95	84	106.5	17	11	14	34	59	7.8	2.0×10^5
10		1735	0.67	42	2.3	32	14	13	125	132	8.0	1.0×10^7
11		1745	0.80	30	2.1	1	0	1	64	34	7.5	2.3×10^6
12		1755	1.30	45	7.2	209	174	152	150	505	7.5	9.0×10^6
13		1715	6.40	60	78.5	53	42	41	193	137	7.5	1.8×10^7
14		1700	0.35	24	0.3	166	77	166	194	402	7.6	1.6×10^7
15		1655	0.41	45	0.6	37	23	15	195	211	7.6	1.4×10^7
16		1710	0.80	24	5.5	327	182	259	168	627	7.5	2.0×10^7
Run No. 6												
1	14 May 69	1901	8.05	60	66.3	969	815	890	380	672	7.2	1.4×10^7
2		1921	3.68	114	90.0	56	16	49	64	122	7.2	4.0×10^7
3		1918	6.88	108	278.0	271	260	267	198	98	7.3	9.0×10^6
4		1947	3.20	60	55.0	71	59	15	90	181	7.4	8.0×10^6
5		1942	3.55	60	55.8	8	0	8	105	157	7.4	1.7×10^7
6		1940	0.05	10	0.07	193	0	168	171	397	7.6	4.9×10^6
7		1925	0.25	56	0.2	86	0	66	216	240	7.4	1.0×10^7
8		1930	2.47	108	69.0	199	143	195	118	265	7.4	1.0×10^7
9		1933	3.00	84	111.0	27	17	15	178	34	7.5	2.5×10^6
10		1930	0.67	42	2.3	156	0	35	151	142	7.5	4.0×10^6
11		1945	0.80	30	2.1	4	4	1	45	25	7.4	4.4×10^5
12		1950	1.30	45	7.2	144	110	100	181	220	7.4	3.0×10^6
13		1912	6.30	60	80.5	100	54	100	122	118	7.3	4.0×10^7
14		1900	0.35	24	0.4	126	109	57	165	172	7.5	7.0×10^6
15		1855	0.36	45	0.4	69	60	72	209	294	7.5	1.6×10^7
16		1910	0.85	24	6.1	179	46	148	225	397	7.4	1.6×10^6

Appendix A (continued)

DRY-WEATHER COMBINED FLOW CHARACTERISTICS

Station Number	Date	Time	Station Flow Depth, ft	Pipe Diameter, in.	Flow, cfs	Total Suspended Solids, mg/l	Volatile Suspended Solids, mg/l	Settleable Solids, mg/l	BOD, mg/l	COD, mg/l	pH	Fecal Coliforms, org/l
Run No. 7												
1	14 May 69	2100	8.00	60	70.6	49	0	25	193	172	7.4	1.0×10^{7}
2		2130	3.53	114	87.5	612	604	572	295	284	7.5	6.0×10^{6}
3		2125	6.78	108	274.0	64	0	43	169	169	7.3	8.0×10^{6}
4		2159	3.10	60	52.5	134	5	118	626	338	7.9	1.4×10^{7}
5		2155	3.40	60	52.6	527	404	506	158	421	7.8	1.2×10^{7}
6		2143	0.02	10	0.03	41	0	18	74	142	7.5	5.7×10^{6}
7		2132	0.31	56	0.3	93	55	88	105	235	7.4	1.4×10^{7}
8		2127	2.57	108	94.5	51	0	41	178	309	7.2	2.0×10^{6}
9		2142	2.40	84	73.4	68	18	67	46	14	7.5	7.5×10^{5}
10		2120	0.72	42	2.6	35	14	72	135	221	7.4	2.4×10^{7}
11		2150	0.85	30	2.3	16	0	6	28	34	7.2	4.4×10^{5}
12		2155	1.45	45	8.9	186	175	35	310	480	7.1	1.6×10^{7}
13		2115	6.30	60	80.5	98	41	75	200	113	7.4	1.2×10^{7}
14		2055	0.45	24	0.5	206	149	190	750	1,882	7.0	1.5×10^{7}
15		2045	0.51	45	0.9	89	5	80	209	225	7.4	2.0×10^{7}
16		2105	0.95	24	7.5	119	36	57	311	451	7.2	3.0×10^{6}
Run No. 8												
1	14 May 69	2300	7.80	60	70.6	52	20	52	140	75	7.4	4.0×10^{7}
2		2330	3.33	114	78.5	93	67	81	133	216	7.3	6.0×10^{7}
3		2326	0.58	108	269.0	50	38	40	196	176	7.0	6.0×10^{7}
4		2358	2.90	60	47.5	91	57	60	163	75	7.0	4.0×10^{7}
5		2354	3.20	60	48.5	152	131	83	38	400	7.2	3.0×10^{7}
6		2328	0.04	10	0.06	147	45	39	150	428	7.2	1.4×10^{7}
7		2323	0.29	56	0.2	117	53	109	70	66	7.2	4.0×10^{7}
8		2318	2.46	108	86.8	66	61	61	215	230	7.0	1.0×10^{8}
9		2344	2.70	84	93.0	111	56	105	26	28	7.4	1.6×10^{6}
10		2309	0.74	42	2.8	80	38	74	700	56	7.2	8.0×10^{7}
11		2335	0.76	30	1.90	53	22	47	33	19	7.3	5.0×10^{6}
12		2343	1.37	45	8.0	142	123	121	61	38	6.6	4.0×10^{7}
13		2339	6.10	60	70.2	352	134	352	75	66	7.2	4.0×10^{7}
14		2250	0.48	24	0.6	50	49	43	163	66	7.2	1.6×10^{7}
15		2245	0.46	45	0.7	97	51	81	203	169	7.1	2.0×10^{7}
16		2300	0.95	24	7.5	161	719	139	225	235	6.8	1.9×10^{7}
Run No. 9												
1	15 May 69	0100	7.40	60	70.6	210	169	188	186	207	7.1	4.0×10^{7}
2		0121	2.93	114	62.0	109	61	86	146	113	6.7	4.0×10^{7}
3		0117	6.18	108	244.0	529	499	481	250	197	6.9	4.0×10^{7}
4		0155	2.10	60	27.5	95	36	82	130	85	7.1	6.5×10^{7}
5		0150	2.70	60	37.1	40	31	25	73	56	7.0	1.6×10^{7}
6		0133	0.03	10	0.045	144	101	135	22	24	7.2	1.0×10^{7}
7		0123	0.31	56	0.3	18	0	14	51	14	7.4	8.0×10^{6}
8		0118	2.17	108	66.8	146	90	47	200	141	6.7	1.6×10^{7}
9		0138	8.10	84	56.5	110	21	96	40	28	7.2	1.4×10^{7}
10		0110	0.63	42	2.0	71	32	66	53	28	7.3	1.2×10^{7}
11		0140	0.78	30	1.96	36	8	36	214	9	7.1	9.2×10^{6}
12		0145	1.27	45	7.0	174	24	159	205	165	7.0	4.0×10^{7}
13		0113	5.60	60	73.5	81	81	81	78	66	7.2	3.9×10^{7}
14		0050	0.42	24	0.5	37	20	31	123	47	7.1	2.0×10^{7}
15		0045	0.43	45	0.2	108	39	94	185	47	7.2	1.0×10^{7}
16		0059	0.82	24	5.8	66	64	63	226	183	6.6	1.6×10^{6}

Appendix A (continued)

DRY-WEATHER COMBINED FLOW CHARACTERISTICS

Station Number	Date	Time	Station Flow Depth, ft	Pipe Diameter, in.	Flow, cfs	Total Suspended Solids, mg/l	Volatile Suspended Solids, mg/l	Settleable Solids, mg/l	BOD, mg/l	COD, mg/l	pH	Fecal Coliforms, org/l
Run No. 10												
1	15 May 69	0300	6.60	60	70.6	34	34	33	208	113	7.2	1.0×10^7
2		0324	1.13	114	9.0	610	439	609	168	71	7.1	4.0×10^7
3		0318	5.38	108	200.0	236	158	220	168	207	6.5	1.6×10^7
4		0352	1.30	60	11.0	128	91	127	128	80	7.0	1.2×10^7
5		0344	2.00	60	22.0	44	23	43	73	49	7.3	2.0×10^7
6		0326		10	0							
7		0320	0.27	56	0.2	6	4	2	216	24	7.1	2.0×10^7
8		0315	1.83	108	47.3	65	42	24	206	132	7.3	2.0×10^7
9		0331	1.70	84	37.3	199	50	195	9	28	7.4	7.5×10^5
10		0309	0.61	42	1.9	9	4	8	42	24	7.0	7.2×10^7
11		0332	0.7	30	1.4	145	104	145	22	28	7.3	1.7×10^6
12		0338	1.07	45	5.0	79	57	77	53	191	7.0	1.5×10^7
13		0313	4.70	60	93.2	28	11	22	111	56	7.0	6.0×10^7
14		0251	0.35	24	0.3	69	25	61	47	19	7.2	1.8×10^7
15		0245	0.41	45	0.6	2	1	2	140	56	7.2	1.2×10^7
16		0300	0.62	24	3.3	117	89	117	203	160	7.2	2.6×10^6
Run No. 11												
1	15 May 69	0510	5.60	60	265.0	33,256	28,684	32,346	1300	4,080	5.6	3.0×10^7
2		0533	1.13	114	9.0	132	87	119	130	99	6.9	3.0×10^7
3		0529	4.28	108	136.5	83	60	76	110	150	7.0	2.0×10^7
4		0559	1.20	60	9.4	28	25	27	165	132	6.8	5.0×10^7
5		0552	1.80	60	18.1	90	36	86	57	94	7.2	3.2×10^7
6		0530	0.04	10	0.05	23	0	23	50	38	7.2	2.0×10^7
7		0522	0.29	56	0.2	123	74	112	66	28	7.3	1.3×10^7
8		0518	1.57	108	35.3	164	0	100	210	127	7.0	2.6×10^7
9		0545	1.10	84	15.4	235	20	2.2	22	28	7.2	1.4×10^6
10		0512	0.60	42	1.8	34	21	24	203	28	7.1	5.5×10^6
11		0534	0.70	30	1.4	53	18	15	78	19	7.1	3.1×10^6
12		0540	1.02	45	4.5	200	170	190	135	191	7.0	4.2×10^5
13		0520	3.60	60	75.5	207	134	169	68	132	7.0	8.0×10^7
14		0452	0.37	24	0.4	182	77	176	155	47	7.2	4.0×10^7
15		0445	0.41	45	0.6	55	28	50	124	24	7.2	1.5×10^7
16		0503	0.67	24	3.9	21	20	9	169	122	7.2	6.0×10^7
Run No. 12												
1	15 May 69	0654	4.5	60	70.2	53	21	52	83	47	7.2	6.0×10^7
2		0706	0.73	114	3.6	73	41	65	168	68	7.2	8.0×10^7
3		0704	3.28	108	84.6	457	350	419	168	437	6.5	1.7×10^7
4		0730	1.30	60	11.0	309	159	253	180	310	7.0	4.0×10^7
5		0726	1.80	60	18.1	195	63	187	130	94	7.1	3.4×10^7
6		0733	0.08	10	0.12	123	81	106	211	179	7.2	1.6×10^7
7		0723	0.35	56	0.3	190	187	65	168	113	7.4	1.6×10^7
8		0720	1.63	108	37.3	46	40	0	185	118	7.2	4.5×10^6
9		0717	1.00	84	12.7	79	21	56	30	19	7.6	6.4×10^5
10		0712	0.75	42	2.9	48	36	42	170	132	6.9	2.7×10^7
11		0742	0.79	30	2.0	80	30	20	35	28	7.4	2.6×10^6
12		0750	1.05	45	4.8	116	66	101	105	94	7.0	2.0×10^6
13		0700	2.7	60	49.2	108	41	105	73	56	7.3	6.0×10^7
14		0655	0.43	24	0.4	231	136	211	78	89	7.5	1.4×10^7
15		0645	0.43	45	0.2	8	8	4	54	47	7.5	4.0×10^7
16		0730	0.68	24	3.3	40	37	25	32	66	7.2	1.0×10^7

Appendix B

WET-WEATHER COMBINED FLOW CHARACTERISTICS

Station Number	Date	Time	Station Flow Depth, ft	Pipe Diameter, in.	Flow, cfs	Total Suspended Solids, mg/l	Volatile Suspended Solids, mg/l	Settleable Solids, mg/l	BOD, mg/l	COD, mg/l	pH	Fecal Coliforms, org/l
Run No. 1												
1	10 Dec 68	1103	5.4	60	>65.0	2,764	2,503	2,719	171	3,360	6.1	2.0×10^7
2												
3		1130	4.48	108	153.0	288	202	261	161	506	6.7	8.0×10^6
4		1130	3.3	60	57.6	484	344	361	133	364	8.25	2.6×10^6
5		1115	3.9	60	62.2	99	99	94	121	300	6.85	2.4×10^7
6		1151	0.6	10	2.2							
7		1150	1.75	56	13.4	296	186	148	145	460	6.8	2.4×10^6
8		1205	3.97	108	218.0	227	164	28	74	387	6.9	2.4×10^6
9		1055	4.5	84	217.0	584	164	404	134	152	7.2	4.4×10^6
10		1139		42		317	196	294	119	316	6.4	2.5×10^6
11		1057	0.8	30	2.05	81	58	81	127	654	7.05	5.4×10^6
12		1114	1.5	45	9.4	104	91	95	195	866	6.65	3.7×10^6
13		1118	3.1	60	61.0	27,490	19,988	27,490	3650	35,000	5.6	1.3×10^8
14		1200	0.4	24	0.4	84	84	26	120	229	6.85	2.0×10^5
15		1129	0.4	45	0.6	53	4	0	113	224	6.55	2.0×10^6
16		1145	0.7	24	4.3	208	208	70	293	641	6.9	9.0×10^6
Run No. 2												
1	10 Dec 68	1445	7.5	60	44.0	1,087	832	783	140	1,100	6.6	1.6×10^7
2												
3		1524	5.48	108	200.0	354	226	315	119	200	7.3	3.0×10^6
4		1500	5.0	60	73.0	44	35	44	97	134	7.45	4.4×10^6
5		1445	5.8	60	92.0	115	38	95	110	139	6.85	9.0×10^7
6		1525	0.2	10	0.4							
7		1515	1.75	56	13.4	197	144	144	105	134	7.10	2.1×10^6
8		1530	3.67	108	188.0	47	26	0	197	234	7.05	2.6×10^6
9		1430	6.4	84	814.0	282	93	197	74	174	8.95	2.0×10^6
10		1515	1.5	42	10.5	270	132	242	130	168	6.75	2.4×10^6
11		1423	1.[illegible]	30	4.9	45	28	35	84	100	6.95	2.5×10^6
12		1435	2.5	45	22.0	136	136	53	142	259	7.05	2.4×10^6
13		1505	5.0	60	85.0	5,022	1,926	4,749	274	4,140	6.15	6.5×10^6
14		1530	0.6	24	0.9	2,280	720	1,768	130	105	10.1	1.0×10^6
15		1442	1.1	45	4.7	350	247	315	62	165	6.65	2.1×10^5
16		1500	0.9	24	6.8	1,379	1,300	500	245	695	6.6	1.4×10^7
Run No. 3												
1	11 Dec 68	0855	4.2	60	67.0	200	200	200	158	238	7.7	5.0×10^6
2												
3		0915	3.28	108	84.5	28	28	22	137	127	7.35	7.5×10^6
4		1025	1.1	60	7.9	160	160	116	185	470	9.65	1.4×10^7
5		1015	1.8	60	18.1	62	62	52	161	228	8.45	1.2×10^8
6		0926	0.1	10	0.2							
7		1035	0.03	56	0.02	2,176	790	2,165	240	337	8.35	3.0×10^4
8		1045	7.87	108	51.6	230	60	119	171	291	7.3	6.6×10^7
9		0955	2.6	84	85.7	54	54	43	119	56	7.38	9.5×10^4
10		0915	1.7	42	6.1	88	88	63	168	227	7.4	9.0×10^7
11		0845	0.7	30	1.6	1,257	665	1,250	125	63	7.65	4.0×10^6
12		0857	1.3	45	7.3	2,207	298	1,924	152	278	7.2	1.5×10^6
13		0907	2.6	60	47.2				165	844	6.9	2.1×10^6
14		0840	0.4	24	0.4	169	169	21	223	405	7.65	2.4×10^6
15		0908	0.3	45	0.3	22	22	22	103	232	8.8	6.0×10^5
16		0910	0	24	0	12	6	5	174	167	7.25	1.0×10^5

Appendix B (continued)

WET-WEATHER COMBINED FLOW CHARACTERISTICS

Station Number	Date	Time	Station Flow Depth, ft	Pipe Diameter, in.	Flow, cfs	Total Suspended Solids, mg/l	Volatile Suspended Solids, mg/l	Settleable Solids, mg/l	BOD, mg/l	COD, mg/l	pH	Fecal Coliforms, org/l
Run No. 1												
1	13 Dec 68	1900	6.6	60	50.5	164	164	164	190	284	7.1	1.0×10^7
2												
3		1935	1.28	108	126.5	4,485	4,264	4,397	5950	3,800	6.9	1.0×10^8
4		2010	3.3	60	56.5	1,804	1,268	1,550	765	9,540	6.0	3.7×10^7
5		2000	3.7	60	58.6	592	506	592	175	370	7.0	4.9×10^7
6		1952	0.0167	10	>0							
7		1947	0.55	56	133.0	103	73	63	195	330	7.0	2.0×10^6
8		1942	2.87	108	117.0	216	173	159	193	380	6.95	3.0×10^7
9		2025	5.0	84	252	806	328	714	125	240	7.4	2.0×10^6
10		1932	1.5	42	10.5	48	40	48	143	248	6.5	4.8×10^7
11		1904	0.9	30	2.6	35	35	25	108	140	7.1	7.0×10^6
12		1916	1.25	45	6.65	86	83	86	238	380	7.0	1.7×10^8
13		1920	4.70	60	93.2	2,822	2,603	2,416	1983	5,610	6.0	8.0×10^8
14		1930	0.60	24	0.95	590	590	590	175	440	6.8	1.0×10^7
15		1945	0.81	45	2.53	61	61	37	150	294	6.8	1.5×10^6
16		1910	0.90	24	6.80	106	106	106	185	367	7.0	1.6×10^6
Run No. 2												
1	13 Dec 68	2155	7.0	60	50.5	249	123	220	177	276	7.0	2.0×10^6
2												
3		2220	6.18	108	245	152	130	135	210	180	7.0	2.0×10^6
4		2245	5.9	60	71.0	1,502	550	1,063	3008	2,410	6.6	5.2×10^7
5		2300	6.6	60	65.0	169	109	169	135	164	7.0	8.0×10^6
6		2243	0.0167	10	>0							
7		2232	2.95	56	32.2	79	79	67	138	124	7.2	1.8×10^7
8		2229	4.77	108	291	53	53	53	160	146	6.9	8.6×10^7
9		2310	7.5	84	320.0	466	75	388	220	196	7.2	
10		2217	2.3	42	22.0	138	31	123	150	94	7.4	3.0×10^7
11		2158	1.10	30	3.70	126	30	125	130	96	7.3	2.6×10^7
12		2209	2.35	45	20.2	282	235	107	215	320	7.1	2.2×10^7
13		2210	4.70	60	93.2	1,666		1,310	239.8	2,500	6.6	1.3×10^8
14		2220	1.00	24	2.43	28	18	28	103	42	7.0	7.6×10^4
15		2228	1.41	45	7.55	16	16	15	110	58	7.0	3.0×10^4
16		2200	1.00	24	8.18	265	265	186	335	559	6.9	4.0×10^6
Run No. 3												
1	14 Dec 68	0910	5.5	60	66.4	95	46	79	191	55	7.7	1.2×10^7
2												1.0×10^7
3		0930	4.18	108	131.0	153	153	141	204	216	7.2	2.4×10^7
4		0950	1.40	60	12.6	105	105	105	218	266	7.4	1.1×10^8
5		0955	2.80	60	39.3	150	150	150	279	242	8.0	6.0×10^7
6		1049	0.016	10	>0							
7		1042	0.35	56	0.51	94	94	83	195	151	7.5	6.0×10^6
8		0932	2.17	108	61.5	30	30	28	328	431	7.2	1.2×10^7
9		1005	2.70	84	92.3	344	63	325	184	74	7.5	4.0×10^6
10		0923	0.70	42	2.5	208	71	178	188	168	7.5	1.3×10^8
11		0905	0.60	30	1.2	4	4	0	150	28	7.7	2.2×10^7
12		0915	1.35	45	7.8	319	208	306	174	158	7.4	5.2×10^4
13		0925	3.70	60	78.0	433	256		393	1,140	7.1	1.2×10^7
14		0927	0.40	24	0.43	41	35	47	135	240	7.8	1.0×10^7
15		0936	0.31	45	0.35	139	40	137	125	120	7.8	4.4×10^7
16		0915	0.80	24	5.5	53	46	53	150	325	7.3	6.0×10^7

Appendix B (continued)

WET-WEATHER COMBINED FLOW CHARACTERISTICS

Station Number	Date	Time	Station Flow Depth, ft	Pipe Diameter, in.	Flow, cfs	Total Suspended Solids, mg/l	Volatile Suspended Solids, mg/l	Settleable Solids, mg/	BOD, mg/l	COD, mg/l	pH	Fecal Coliforms, org/l
Run No. 1												
1	18 Jan 69	1600	5.8	60	66.6	419	88	380	275	144	6.80	2.3×10^6
2		1632	5.53	114	186.0	182	61	165	192	163	6.65	2.0×10^6
3		1628	6.48	108	250.0	424	306	419	178	114	7.10	4.0×10^6
4		7645	5.3	60	74.5	159	65	91	202	169	6.90	1.5×10^6
5		1705	6.4	60	138.0	118	28	108	164	101	7.05	$1/8 \times 10^6$
6		1645		10	>0							
7		1634	3.55	56	41.0	270	178	204	335	479	6.60	7.5×10^6
8		1629	4.77	108	291.0	502	311	480	241	513	7.05	5.9×10^6
9		1720	6.9	84	303.0	441	66	84	474	127	7.7	2.7×10^6
10		1620	3.07	42	30.0	373	288	364	723	87	6.6	6.0×10^5
11		1556	1.6	30	6.8	159	83	114	200	50	7.20	3.0×10^5
12		1611	3.35	45	29.8	940	698	867	497	836	6.55	2.6×10^6
13		1620	3.7	60	78.0	244	123	235	383	139	6.70	3.0×10^6
14		1648	1.20	24	2.4	129	44	110	142	86	6.95	1.7×10^6
15		1657	2.61	45	18.7	98	22	82	150	37	7.80	7.2×10^5
16		1670	1.0	24	8.2	1,137	803	1,005	682	1,566	7.00	1.8×10^7
Run No. 2												
1	18 Jan 69	1905	4.9	60	69.5	262	70	206	332	133	6.40	3.6×10^6
2		1938	4.13	114	117.0	144	41	110	112	66	6.55	2.8×10^6
3		1933	5.58	108	210.0	386	137	371	367	192	7.40	2.5×10^6
4		1950	4.4	60	78.6	308	0	241	307	245	6.30	7.4×10^6
5		2000	4.9	60	70.0	72	35	61	109	68	6.95	3.6×10^6
6		1945	0	10	0							
7		1936	2.05	56	17.8	241	162	206	230	287	6.35	5.3×10^6
8		1932	4.12	108	233.0	186	186	111	258	257	6.75	1.6×10^6
9		2010	6.6	84	310.0	121	23	100	422	43	6.50	1.6×10^6
10		1923	1.57	42	11.9	53	28	34	186	53	6.65	4.0×10^6
11		1906	0.8	30	2.1	401	54	375	260	123	6.90	2.8×10^6
12		1914	2.95	45	26.8	396	250	346	784	476	6.85	5.6×10^6
13		1925	3.0	60	58.3	1,057	587	893	2055	1,310	6.35	7.0×10^6
14		1940	0.61	24	1.0	73	42	70	175	71	6.55	2.4×10^6
15		2000	0.90	45	2.8	33	19	28	150	41	6.50	4.0×10^5
16		1918	1.47	24	14.5	191	119	138	220	298	6.65	4.8×10^6
Run No. 3												
1	19 Jan 69	0845	6.5	60	74.5	227	42	143	348	65	6.55	9.6×10^5
2		0910	5.63	114	219.0	77	33	64	144	39	6.75	5.1×10^5
3		0907	6.08	108	237.0	80	26	56	198	37	6.50	4.0×10^6
4		0923	3.6	60	66.7	106	34	102	168	72	7.60	1.1×10^6
5		0930	4.1	60	56.8	70	35	64	185	62	6.60	1.6×10^7
6		0918	0.1	10	0.14	34	30	22	176	82	6.75	4.9×10^6
7		0910	2.15	56	19.3	82	37	78	260	73	6.95	5.9×10^5
8		0906	4.67	108	281.0	66	26	44	175	59	7.00	8.9×10^5
9		0938	4.8	84	235.0	865	48	660	190	46	6.20	5.9×10^5
10		0859	1.27	42	8.0	59	26	59	139	27	6.55	4.0×10^6
11		0840	1.0	30	3.1	72	22	59	205	39	6.90	2.0×10^4
12		0852	3.25	45	29.2	2,096	2,056	2,078	148	110	6.30	1.4×10^5
13		0900	3.7	60	78.0	203	0	174	377	55	6.65	1.7×10^6
14		0918	0.80	24	1.6	64	28	40	127	51	7.05	2.6×10^6
15		0928	1.09	45	4.1	42	8	36	197	48	7.00	1.2×10^6
16		0952	1.57	24	15.6	52	35	38	160	72	6.95	1.4×10^4

Appendix B (continued)

WET-WEATHER COMBINED FLOW CHARACTERISTICS

Station Number	Date	Time	Station Flow Depth, ft	Pipe Diameter, in.	Flow, cfs	Total Suspended Solids, mg/l	Volatile Suspended Solids, mg/l	Settleable Solids, mg/l	BOD mg/l	COD mg/l	pH	Fecal Coliforms, org/l
Run No. 1												
1	5 Feb 69	2020	6.80	60	61.40	179	60	152	157	119	7.3	2.9×10^6
2		2035	3.53	114	88.00	88	22	88	92	99	7.2	3.8×10^6
3		2050	5.68	108	217.00	211	144	180	95	142	7.4	5.3×10^6
4		2105	4.50	60	79.50	139	65	119	68	159	6.9	5.6×10^6
5		2115	4.70	60	70.30	64	30	64	61	65	7.0	1.6×10^6
6		2059	0.04	10	0.07	66	7	57	166	144	7.8	1.6×10^7
7		2050	2.05	56	17.70	109	29	95	54	240	7.3	2.3×10^6
8		2045	4.22	108	237.00	140	56	127	808	160	7.2	1.9×10^6
9		2125	5.30	84	268.00	341	62	252	206	111	7.1	3.0×10^5
10		2038	1.27	42	8.05	86	15	82	77	73	7.5	1.4×10^6
11		2018	0.70	30	1.58	126	15	125	169	62	7.6	8.8×10^5
12		2027	2.15	45	17.60	592	349	558	350	686	7.1	1.2×10^2
13		2040	4.50	60	92.60	834	455	817	276	1,230	6.8	3.2×10^7
14		2057	0.48	24	0.61	35	35	26	100	91	6.8	1.4×10^7
15		2107	0.58	45	1.16	27	27	27	71	39	7.5	2.5×10^6
16		2043	1.27	24	12.00	344	279	324	278	500	6.9	1.5×10^7
Run No. 2												
1	5 Feb 69	2317	6.10	60	61.40	105	25	83	131	122	7.35	1.1×10^6
2		2345	4.43	114	132.00	107	51	103	41	52	7.2	1.7×10^6
3		2340	5.38	108	201.00	82	39	74	89	420	6.75	1.2×10^6
4		2355	4.60	60	80.00	109	39	103	69	94	7.6	3.4×10^6
5	6 Feb 69	0005	4.80	60	70.00	76	58	36	57	64	7.0	1.9×10^6
6		0001	0.04	10	0.07	31	6	22	51	93	7.5	1.6×10^7
7	5 Feb 69	2350	1.95	56	16.00	54	27	52	62	48	7.0	6.0×10^6
8		2346	4.07	108	223.00	110	82	106	132	176	6.9	1.6×10^6
9	6 Feb 69	0015	6.30	84	308.00	90	26	0	140	57	7.6	5.7×10^5
10	5 Feb 69	2336	1.07	42	5.80	31	0	31	46	34	7.4	1.7×10^6
11		2323	0.50	30	0.82							
12		2331	2.45	45	21.30	361	185	334	336	500	7.4	2.0×10^7
13		2335	3.50	60	72.70	562	276	523	353	705	6.9	4.6×10^6
14		2345	0.53	24	0.75	36	23	35	35	45	6.9	1.6×10^6
15		2353	0.84	45	2.45	16	15	10	45	48	7.6	3.0×10^6
16		2333	1.30	24	12.33	2,460	18	2,404	200	363	7.2	8.0×10^4
Run No. 3												
1	6 Feb 69	1430	6.70	60	70.50	253	63	181	51	126	6.9	6.0×10^6
2		1500	2.23	114	36.10	127	113	113	69	203	6.9	1.7×10^7
3		1455	4.78	108	164.00	127	110	104	100	266	7.1	1.6×10^7
4		1515	2.40	60	34.80	137	71	118	115	247	7.7	1.8×10^7
5		1528	2.90	60	41.60	107	66	96	98	174	7.3	8.4×10^6
6		1511	0.21	10	0.42	38	37	9	100	116	7.4	2.3×10^7
7		1458	0.15	56	0.09	182	129	136	116	299	7.4	6.6×10^6
8		1450	2.27	100	72.00	98	98	70	186	285	7.2	7.0×10^6
9		1533	3.80	84	165.00	191	39	45	176	41	7.6	5.4×10^6
10		1442	0.77	42	3.02	61	50	54	88	139	7.6	2.6×10^7
11		1429	0.50	30	0.81	225	193	211	82	156	7.5	2.0×10^7
12		1436	1.55	45	10.00	17	11	0	174	283	7.3	2.8×10^6
13		1440	4.40	60	91.50	660	430	602	117	581	7.1	1.3×10^7
14		1500	0.35	24	0.32	142	95	142	103	461	7.4	2.5×10^7
15		1508	0.52	45	0.94	74	47	72	92	125	7.6	3.4×10^6
16		1453	1.27	24	11.92	608	572	571	258	686	7.4	2.6×10^6

Appendix B (continued)

WET-WEATHER COMBINED FLOW CHARACTERISTICS

Station Number	Date	Time	Station Flow Depth, ft	Pipe Diameter, in.	Flow, cfs	Total Suspended Solids, mg/l	Volatile Suspended Solids, mg/l	Settleable Solids, mg/l	BOD, mg/l	COD, mg/l	pH	Fecal Coliforms, org/l
Run No. 1												
1	23 Feb 69	0225	6.60	60	54.3	215.7	71.4	208.5	74	258.8	7.1	9.9×10^{6}
2		0300	4.73	114	149.5	135.0	55.0	133.4	30	142.6	6.9	8.0×10^{5}
3		0255	7.58	108	304.0	34.5	19.0	25.7	171	140.7	7.0	3.8×10^{6}
4		0315	8.40	60	82.0	106.0	63.0	80.4	25	195.2	6.9	1.0×10^{7}
5		0328	8.30	60	103.0	218.0	150.0	198.0	34	63.6	7.2	2.7×10^{6}
6		0247	0	10	0							
7		0241	2.65	56	2[illegible].2	67.0	49.0	30.2	62	136.2	7.0	1.5×10^{6}
8		0235	4.27	108	242.0	56.0	51.0	34.4	75	195.2	7.5	1.2×10^{6}
9		0340		84								
10		0227	2.47	42	24.0	146.0	65.8	139.6	2160	127.1	6.9	1.1×10^{7}
11		0209	1.10	30	3.7	72.1	21.0	58.9	44	95.3	7.1	1.2×10^{6}
12		0219	1.75	45	12.4	308.5	128.5	298.1	237	431.3	7.0	7.7×10^{6}
13		0245	5.70	60	87.0	113.8	52.3	80.2	69	572.0	6.7	1.6×10^{7}
14		0255	0.90	24	2.0	39.3	9.3	[illegible].7	59	86.3	7.0	3.3×10^{6}
15		0305	2.43	45	16.8	114.0	28.0	[illegible]3.2	30	118.0	7.5	5.5×10^{6}
16		0245	0.65	24	3.7	82.5	25.0	71.7	71	417.7	7.1	3.0×10^{4}
Run No. 2												
1	23 Feb 69	0515	4.60	60	70.7	72.0	33.0	69.2	42	86.3	7.5	3.8×10^{6}
2		0533	4.03	114	111.5	91.0	17.0	60.6	85	86.3	7.7	2.2×10^{6}
3		0528	4.48	108	148.5	307.0	135.0	297.0	172	109.0	7.0	1.3×10^{6}
4		0543	3.40	60	59.8	105.0	45.0	95.0	34	72.6	7.2	1.1×10^{6}
5		0550	3.80	60	60.3	51.4	24.0	45.0	53	68.1	7.3	3.6×10^{6}
6		0545	0.04	10	0.07	32.7	32.7	22.7	59	217.9	7.3	1.3×10^{7}
7		0537	1.15	56	5.9	30.0	30.0	31.2	219	168.0	7.4	5.9×10^{5}
8		0533	3.27	108	149.5	151.2	112.0	142.4	73	190.7	7.5	6.6×10^{5}
9		0600	4.30	84	201.0	136.0	16.0	22.0	44	34.5	7.2	7.7×10^{5}
10		0525	0.77	42	3.0	21.6	14.2	17.2	44	36.2	6.6	2.7×10^{6}
11		0510	0.60	30	1.2	17.5	15.0	6.3	53	13.6	7.5	9.9×10^{5}
12		0520	1.85	45	13.7	193.0	133.0	189.4	73	326.9	6.6	2.7×10^{5}
13		0520	2.50	60	43.5	152.0	73.0	134.8	253	254.2	7.2	4.0×10^{6}
14		0535	0.42	24	0.5	17.0	13.0	14.2	41	31.8	7.0	7.1×10^{6}
15		0542	0.56	45	1.1	33.0	14.0	31.4	44	36.3	7.3	8.8×10^{5}
16		0525	1.00	24	8.2	91.0	57.7	85.4	79	168.0	6.9	4.0×10^{4}
Run No. 3												
1	23 Feb 69	1655	7.70	60	66.0	38.4	25.4	28.4	72	81.7	7.0	1.4×10^{8}
2		1717	3.33	114	78.3	22.0	22.0	13.6	65	217.9	7.0	1.1×10^{7}
3		1713	6.58	108	263.0	59.0	58.0	30.6	71	240.6	7.0	9.4×10^{7}
4		1727	2.80	60	44.8	62.6	62.6	51.6	64	295.1	6.9	2.0×10^{8}
5		1730	3.10	60	46.0	6.0	5.0	1.2	63	122.6	7.1	6.6×10^{6}
6		1721	0.04	10	0.07	11.0	7.0	7.0	46	18.2	6.9	8.0×10^{6}
7		1715		56		100.0	73.6	98.0	29	490.4	6.8	1.0×10^{8}
8		1711	2.27	108	74.0	207.5	162.0	201.9	69	317.8	7.1	7.0×10^{6}
9		1740	3.90	84	174.0	50.8	32.3	46.4	68	31.8	7.0	5.7×10^{5}
10		1705	0.57	42	1.7	44.4	40.0	41.6	69	168.0	7.2	6.6×10^{7}
11		1650	0.50	30	0.8	11.5	1.5	10.7	42	18.2	7.3	1.4×10^{6}
12		1656	1.35	45	7.8	397.0	360.0	321.8	29.5	723.7	6.8	5.5×10^{6}
13				60								
14		1708	0.35	24	0.3	100.0	37.0	81.6	60	199.8	7.2	2.2×10^{7}
15		1715	0.36	45	0.4	35.8	22.5	10.6	253	269.7	7.1	1.0×10^{8}
16		1701	0.90	24	6.8	149.0	149.0	130.6	210	566.4	7.0	1.6×10^{6}

Appendix B (continued)

WET-WEATHER COMBINED FLOW CHARACTERISTICS

Station Number	Date	Time	Station Flow Depth, ft	Pipe Diameter, in.	Flow, cfs	Total Suspended Solids, mg/l	Volatile Suspended Solids, mg/l	Settleable Solids, mg/l	BOD,mg/l	COD,mg/l	pH	Fecal Coliforms, org/l
Run No. 1												
1	5 Apr 69	0440	6.00	60	50.7	156	142	150	383	309	6.7	1.8×10^7
2		0503	2.63	114	49.7	75	58	73	250	91	6.6	3.0×10^6
3		0458	4.68	108	159.0	137	132	125	277	141	6.6	3.8×10^6
4		0515	3.20	60	55.2	820	634	737	1346	1,507	6.8	1.0×10^7
5		0527	3.50	60	54.7	192	140	161	220	172	6.9	5.0×10^6
6		0522	0	10	0							
7		0514	0.35	56	0.5	52	52	45	76	77	6.7	2.4×10^6
8		0509	2.47	108	87.5	242	242	233	306	191	7.0	2.0×10^6
9		0535	4.70	84	230.0	111	60	30	284	82	7.3	1.0×10^7
10		0500	1.17	42	6.9	36	36	32	185	50	6.5	6.0×10^6
11		0445	0.60	30	1.2	180	59	168	71	136	6.9	3.2×10^6
12		0453	1.15	45	5.7	548	414	460	1500	636	6.6	9.0×10^6
13		0453	3.90	60	82.6	1,570	1,570	1,553	>2000	15,799	5.3	8.0×10^7
14		0515	0.49	24	0.6	17	17	17	234	36	7.1	1.8×10^6
15		0525	0.75	45	1.9	15	15	15	74	36	6.8	1.1×10^6
16		0504	0.73	24	4.6	246	223	218	1850	291	6.7	1.4×10^6
Run No. 2												
1	5 Apr 69	0735	5.90	60	70.7	50	39	44	455	118	6.0	1.2×10^7
2		0758	2.43	114	42.7	65	48	65	200	73	6.9	2.6×10^7
3		0755	4.28	108	136.5	706	666	704	84	751	6.2	1.4×10^7
4		0810	2.30	60	32.2	913	705	845	920	899	5.1	2.2×10^7
5		0815	2.80	60	39.4	163	121	128	300	265	6.7	4.0×10^7
6		0814	0	10	0							
7		0806	0.15	56	0.1	36	27	36	252	100	7.0	4.0×10^6
8		0800	2.07	108	61.2	91	91	52	70	123	6.6	4.5×10^6
9		0823	4.10	84	186.5	200	138	142	402	59	7.0	4.0×10^6
10		0752	1.07	42	5.8	96	96	79	262	191	6.9	4.8×10^6
11		0735	0.5	30	0.8	76	40	76	85	50	6.7	4.0×10^5
12		0745	1.05	45	4.8	232	220	150	626	476	6.7	3.0×10^6
13		0750	3.70	60	78.0	1,050	795	1,010	2000	2,724	5.5	4.0×10^7
14		0750	0.55	24	0.3	28	28	11	223	45	6.8	6.0×10^6
15		0757	0.75	45	1.9	32	32	26	80	45	6.9	3.6×10^6
16		0737	0.73	24	4.6	625	487	412	281	636	6.4	1.7×10^7
Run No. 3												
1	5 Apr 69	1817	6.40	60	66.0	130	97	115	192	104	6.4	8.0×10^6
2		1840	2.23	114	36.0	90	88	77	340	91	7.1	1.0×10^7
3		1836	5.28	108	193.5	168	165	135	80	259	6.6	2.4×10^7
4		1905	1.60	60	16.6	111	94	103	80	177	6.4	2.4×10^7
5		1910	2.00	60	22.0	70	70	53	73	204	6.7	8.0×10^7
6		1856	0	10	0							
7		1848	0.05	56	0.1	123	90	107	75	186	6.7	2.0×10^6
8		1845	1.67	108	40.0	241	221	210		354	6.5	1.2×10^7
9		1980	3.20	84	123.0	223	47	223	267	45	7.0	5.2×10^6
10		1835	0.67	42	2.2	52	48	32	326	127	6.6	3.0×10^7
11		1823	0.50	30	0.8	24	24	12	75	14	7.0	2.6×10^6
12		1830	1.35	45	7.8	382	382	362	82	632	6.7	1.0×10^7
13		1830	4.6	60	93.0	62	48	0	346	113	6.0	4.2×10^7
14		1836	0.32	24	0.4	142	113	130	347	113	6.0	9.8×10^7
15		1845	0.40	45	0.5	24	16	24	84	64	7.0	9.6×10^6
16		1823	1.13	24	9.9	661	648	637	68	863	7.0	4.0×10^6

1 *Accession Number*

W

2 *Subject Field & Group*

05 F

SELECTED WATER RESOURCES ABSTRACTS
INPUT TRANSACTION FORM

5 *Organization*

Envirogenics Company, a Division of Aerojet-General Corporation
El Monte, California

6 *Title*

Urban Storm Runoff and Combined Sewer Overflow Pollution

10 *Author(s)*

Feuerstein, D. L.

King, R. W.

Grimm, A.

16 *Project Designation*

FWQA Program 11024 FKM Contract #14-12-197

21 *Note*

22 *Citation*

Water Pollution Control Research Series 11024 FKM 12/71

23 *Descriptors (Starred First)*

Combined sewers, combined sewage, water pollution control, wastewater treatment systems, waste assimilative capacity, surface runoff.

25 *Identifiers (Starred First)*

27 *Abstract*

A general method was developed to assess, primarily from readily available precipitation and wastewater quality data, the extent of water pollution occurring from storm water runoff and combined sewer overflows in an urban area, and is applied to Sacramento, California. Systems for the control and treatment of these wastewaters are developed and evaluated. The least costly system to adequately protect the receiving waters from storm water runoff and combined sewer overflows would retain the combined sewers for the conveyance of combined sewage during wet-weather flow conditions. Facilities would also be required for the treatment of existing separated storm water flows. Total annual cost for this system was estimated to be $6.99 million. A slightly more costly system ($7.09 million) incorporating complete sewer separation of sanitary sewage and storm water runoff is recommended to the City of Sacramento. The similarity in annual costs for the separated sewer and the combined sewer systems results from the requirement for major enlargement of the existing combined sewer system to adequately convey anticipated combined sewage flows. In areas where existing combined sewer capacities would not be grossly inadequate, the separation of combined storm water runoff and sanitary sewage flows to achieve receiving water quality objectives would appear unwarranted, due to the high cost of constructing new conveyance facilities and the probable requirement to treat separated storm water runoff, since its quality is not substantially different from that of sanitary sewage.

Abstractor D. L. Feuerstein

Institution Aerojet-General Corporation

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11023 --- 09/67	Demonstrate Feasibility of the Use of Ultrasonic Filtration in Treating the Overflows from Combined and/or Storm Sewers
11020 --- 12/67	Problems of Combined Sewer Facilities and Overflows, 1967 (WP-20-11)
11023 --- 05/68	Feasibility of a Stabilization-Retention Basin in Lake Erie at Cleveland, Ohio
11031 --- 08/68	The Beneficial Use of Storm Water
11030 DNS 01/69	Water Pollution Aspects of Urban Runoff, (WP-20-15)
11020 DIH 06/69	Improved Sealants for Infiltration Control, (WP-20-18)
11020 DES 06/69	Selected Urban Storm Water Runoff Abstracts, (WP-20-21)
11020 --- 06/69	Sewer Infiltration Reduction by Zone Pumping, (DAST-9)
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11022 DMU 07/70	Combined Sewer Regulator Overflow Facilities
11024 EJC 07/70	Selected Urban Storm Water Abstracts, July 1968 - June 1970
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11023 --- 08/70	Retention Basin Control of Combined Sewer Overflows
11023 FIX 08/70	Conceptual Engineering Report - Kingman Lake Project
11024 EXF 08/70	Combined Sewer Overflow Abatement Alternatives - Washington, D.C.